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FORMULAS AND TABLES

FOR

ARCHITECTS AND ENGINEERS

IN

CALCULATING THE STRAINS AND CAPACITY

OF

STRUCTURES IN IRON AND WOOD,

BY

F. SCHUMANN, C. E.

ILLUSTRATED WITH MORE THAN THREE HUNDRED DIAGRAMS, DESIGNED AND
ENGRAVED ESPECIALLY FOR THIS WORK BY J. C. LYONS.

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THIS VOLUME
IS
RESPECTFULLY DEDICATED
TO
A. B. MULLETT,
SUPERVISING ARCHITECT OF THE U. S. TREASURY DEPARTMENT,
BY THE AUTHOR.

P R E F A C E .

The following collection of Formulas and Tables is intended as a ready reference for Architects and Engineers in computing the Strains and Strength of Structures in Iron or Wood. The author has attempted to give concisely all the necessary data required for the calculations. The object is, to give a compilation and transformation into simple formulas and tables, *adapted to practice*, of matter contained in the works of *Weisbach, Rankine, Rebhahn, Ritter, Breyman, Gordon, Brandt, Moll & Reuleaux, Laissle & Schübler, Fairbairn*, and others.

Among other matter may be particularly mentioned the tables giving the capacity of rolled and cast-iron beams, and the extensive table of formulas for the Moment of Inertia and Resistance of various sections. The formulas and constants for ascertaining the Strains in Roof and other Trusses will be found quite complete, extending to all the different forms or systems in general use. Under "Miscellaneous" is given necessary formulas for the calculation of Lines, Areas, and Solids; also tables of Natural Sines, Cosines, Tangents, Cotangents, Secants, and Cosecants, Area and Circumference of Circles, Weight of Materials, &c.

WASHINGTON, D. C., *March*, 1873.

E R R A T A.

On page 4, 10th line from bottom, read $\frac{30}{100}$ instead of 30.

On page 4, 10th line from bottom, read 10.0036 instead of 10.036.

On page 4, 14th, 15th, and 16th lines from bottom, read $\frac{a}{100}$ instead of a .

On page 32, *Fig. 70*, insert l = distance between supports.

On page 34, *Fig. 72*, insert l = distance between supports.

On page 34, *Fig. 74*, insert l = length of beam.

On pages 38 and 39 w = total weight of beam between supports.

On page 39, 5th line from top, read 1099000 instead of 1000000.

On page 39, 5th line from top, read 1754 instead of 1757.

On pages 144 and 145, in formulas for H_n , change places of last minus sign with foregoing plus sign. (See 13th line from top.)

Page 145, lines 1 to 7 from bottom,	}	Change places of C and T under strains in <i>Figs.</i> 225, 226, 227, and 228.
Page 146, lines 1 to 3 from top,		
Page 146, lines 13 to 22 from top,		

On page 149, 1st line from bottom, read $\frac{lw}{N} - \frac{D}{H-D}$ instead of $\frac{lw}{N}$.

On page 197, 7th line from bottom, read 3.14159 instead of 1.14159.

On page 204, 1st line from bottom, read $A + A$, instead of AA .

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FORMULAS AND TABLES

FOR

ARCHITECTS AND ENGINEERS.

Summary of Definitions and General Principles.

EXTERNAL FORCES are those forces (loads, &c.) which cause or tend to cause the rupture of a structure.

INTERNAL FORCES are those forces which resist the external forces; when one balances the other, the structure is said to possess *Stability*.

EXTERNAL FORCES.

Compressive strain.
Tensional strain.
Shearing strain.
Cross-breaking strain.

INTERNAL FORCES.

Resistance to Compression.
Resistance to Tension.
Resistance to Shearing.
Resistance to Cross-breaking.

COMPRESSION causes the material to fail by *crushing* or *buckling*, or both.

RESISTANCE to direct Crushing: In case pillars, blocks, struts, or rods, along which the strains act, are not so long in proportion to their diameter as to have a tendency to give way by bending sideways. This includes—

Stone and brick pillars and blocks, of ordinary proportions;

Pillars, struts, and rods of cast iron, in which the length is not more than five times the diameter, approximately;

Pillars, struts, and rods of wrought iron, in which the length is not more than ten times the diameter, approximately;

Pillars, struts, and rods of dry timber, in which the length is not more than twenty times the diameter.

Let W = Crushing load in lbs.

C = Ultimate resistance of material to crushing in lbs. per square inch.

A = Sectional area of pillar, &c., in square inches.

Then will $W = A \times C$; and $A = \frac{W}{C}$

TENSION, causes the material to be torn asunder.

Resistance of bars, &c., to tearing: the ultimate strength of a bar (to tearing) is: when

T = Ultimate resistance of the material to tearing, in lbs. per square inch.

W = Tearing load in lbs.

A = Sectional area of bar, in square inches.

Then will $W = A \times T$; and $A = \frac{W}{T}$

SHEARING causes the fibres of the material to be shorn by each other; when a bolt pulls out of its nut, the threads of the screw are said to be sheared.

Resistance of bars, bolts, &c., when sheared at one place, is: when

S = Ultimate resistance of material to shearing, in lbs. per square inch.

W = Shearing load in lbs.

A = Sectional area of bar, &c., in square inches.

Then will $W = A \times S$; and $A = \frac{W}{S}$

CROSS-BREAKING a beam, &c., supported at one or both ends, with a force at right angles to its length, sufficient to cause rupture, is said to be broken across.

Resistance to cross-breaking is the resistance of the material to compression, tension, and shearing combined; ———.

The flanges or booms, in beams or trusses, resist the bending moment, or moment of rupture.

The web or braces, in beams or trusses, resist the shearing forces.

CAPACITY means the load or pressure a structure is capable of sustaining with safety.

DEFLECTION is the displacement of a beam from a horizontal, caused by its own weight or the applied load, or both.

CAMBER is given a beam to counter balance the deflection, so that the beam may be horizontal when loaded; the camber should be equal to the computed deflection.

To find the effect of combining several loads on one beam, whose separate actions are known: for each cross section, the shearing force is the sum of the shearing forces, and the bending moment the sum of the bending moments, which the loads would produce separately.

When a load on a structure is partly static and partly moving, multiply each part of the load by its proper factor of safety, and

add together the products: the sum will be the load to which the structure is to be adapted.

For a Bridge with two platforms, one carrying a road and the other a railway, those two loads are to be combined.

Of Iron Ties, Struts, and Beams.

In designing ordinary structures of wrought iron, it saves time and expense to use iron bars of such forms of cross section as are usually to be met with in the market. An engineer should avoid introducing new sections for bars into his designs, except when, by so doing, some important purpose is to be served, or some decided advantage to be gained. The most common forms of rolled bars is shown by the following enumerated figures:

No. of figure.	Name of Form.	Applicable for—
4	Square iron.....	Ties.
13	Round iron.....	Ties, bolts, and rivets.
2	Flat iron.....	Ties.
29	I or double T-iron.....	Beams, rafters, and struts.
30	Channel iron.....	Rafters and struts.
37	T-iron.....	Rafters and struts.
47	L or angle iron.....	Various purposes.
1	Deck Beam.....	Beams and rafters.

Avoid the use of cast iron for ties, also trussed cast-iron beams.

When a member of a structure acts alternately as a strut and as a tie, it must have sufficient total sectional area, and sufficient stiffness, to resist the greatest compressive strain that can act, and sufficient effective sectional area to resist the greatest tensional strain which can act along it.

Let all pins, bolts, rivets, &c., exposed to a shearing strain, fit tight in its hole or socket.

Roof trusses, the divisions of a rafter, and also the struts, may be considered as hinged at the ends.

In members under a compound strain, as for instance a trussed beam with a distributed load, be careful to take into account the additional compression, caused by the bending moment.

The best distribution of the material in a section of a cast-iron beam, for cross-breaking, is that $\frac{T}{s} = \frac{C}{s'}$; or $\frac{s'}{s} = \frac{C}{T}$

When T = Ultimate resistance of the material to tension.

C = Ultimate resistance of the material to compression.

s = Distance from neutral axis to most extended fibres.

s' = Distance from neutral axis to most compressed fibres.

That is, the fibres most in tension should be nearest the neutral axis of beam.

In wrought-iron beams, the section may be made alike above and below the neutral axis.

THE MODULUS OF RUPTURE should be applied to beams with full section, or beams with continuous web only; for all open web beams, or beams with very thin web, the resistance of the material to crushing or tearing, respectively, must be used instead.

EXPANSION AND CONTRACTION of long beams, which arise from the changes of atmospheric temperature, are usually provided for by supporting one end of each beam on rollers of steel or hardened cast iron. The following table shows the proportions in which the length of a bar of certain materials is increased by an elevation of temperature from the melting point of ice (32° Fahr., or 0° Centigrade) to the boiling point of water under the mean atmospheric pressure, (212° Fahr., or 100° Cent.;) that is, by an elevation of 180° Fahr., or 100° Cent.:

METALS.		EARTHY MATERIALS.	
Brass.....	0.00216	Brick, common.....	0.00355
Bronze.....	0.00181	Brick, fire.....	0.00050
Copper.....	0.00184	Cement.....	0.00140
Cast iron.....	0.00111	Glass, average.....	0.00090
Wrought iron.....	0.00120	Granite.....	0.00085
Tin.....	0.00225	Marble.....	0.00087
Zinc.....	0.00294	Sandstone.....	0.00105
Lead.....	0.00290	Slate.....	0.00104

Reference.

- Let u = Value given in above table, for a certain material.
 l = Length of a bar at 0° Centigrade,
 and l_{\prime} = its length at a given number of degrees Centigrade.
 a = Given number of degrees, at which l_{\prime} is required.
 A = Superficial area of a plate;
 and A_{\prime} = its area at a given number of 0° C.
 B = Cubic contents of a body,
 and B_{\prime} = its contents at a given number of 0° C.

$$\begin{aligned}\text{Then will } l_{\prime} &= l (1 + a u); \\ A_{\prime} &= A (1 + 2 a u); \\ B_{\prime} &= B (1 + 3 a u).\end{aligned}$$

Example: A bar of wrought iron 2 inches square, is 10 feet long at a temperature of 0° Centigrade; what is its length at an increased temperature of 30° ?

$$\text{Ans: } l_{\prime} = 10 (1 + 30 \times 0.00120) = 10.036 \text{ feet.}$$

THE NEUTRAL AXIS, in a cross section of a beam, is that layer of fibres which are neither in compression or tension, by the action of a load on the beam; it always passes through the centre of gravity of the section: provided the limits of elasticity of the material is not exceeded. A beam supported at both ends, with a load acting perpendicular between the supports, will cause the fibres above the neutral axis to be compressed, and those below, extended: the farther from the fibres to the neutral axis, the greater the stress.

Under MOMENT OF INERTIA of a cross section, may be understood: an internal force at rest; a static force resisting an external force; it means the sum of all the area elements, multiplied by the square of their perpendicular heights from the neutral axis of the section. The moment of inertia, which we have denoted with I , depends on the form and dimensions of the cross section, and is expressed in square inches.

MOMENT OF RESISTANCE of a cross section is that static force resisting an external force of compression or tension; it is equal to the moment of Inertia divided by the distance of the most extended or compressed fibres, respectively, from the neutral axis.

MOMENTS OF INERTIA AND RESISTANCE OF VARIOUS SECTIONS.

To find the moment of inertia of any given cross section—

FIRST. Divide the section into as many simple figures as possible. (See diagram, fig. 1.)

SECOND. Find the moment of inertia of each of the simple figures about its own neutral axis, and insert the value in the following formula:

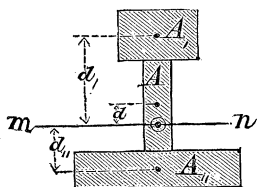
Reference.

Letters $A, A', A'',$ = area of simple figure, respectively; and
 $d, d', d'',$ = its distance from its centre of gravity
 to that of the whole section.

$i, i', i'',$ = moment of inertia of simple figures, respectively.

For neutral axis see centre of gravity.

Fig. 1.



Formula.

$$I = (i + d^2A) + (i' + d'^2A') + (i'' + d''^2A'') + \&c., = \text{moment of inertia of whole section.}$$

MOMENTS OF INERTIA I AND MOMENTS OF RESISTANCE $\frac{I}{s}$

Reference.

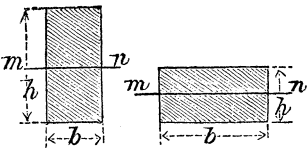
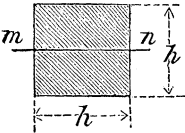
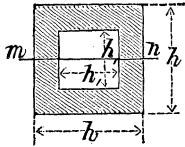
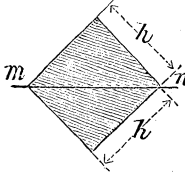
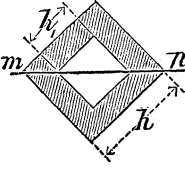
$m - n$ = neutral axis of section.

r = radius.

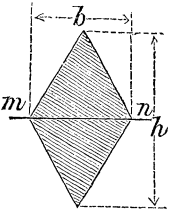
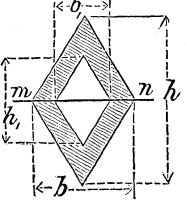
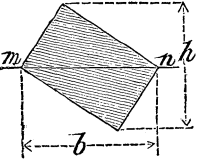
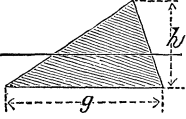
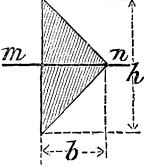
s = distance from neutral axis to most compressed or extended fibres.

$b, h, \&c.$ = dimensions.

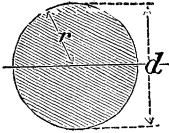
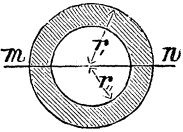
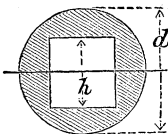
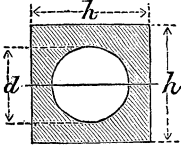
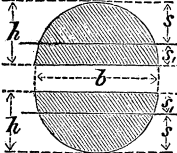
A = area.

No. of Section.	No. of Figure.	Form of Section.
I.	2 and 3	
II.	4	
III.	5	
IV.	6	
V.	7	

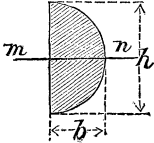
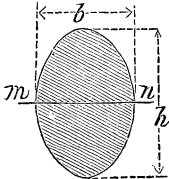
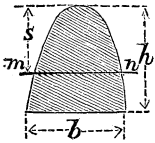
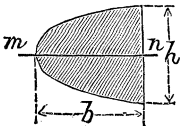
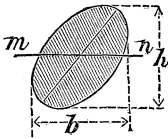
Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{12} bh^3 = \frac{1}{12} Ah^2$	$\frac{bh^2}{6}$
$\frac{1}{12} h^4 = \frac{1}{12} Ah^2$	$\frac{h^3}{6}$
$\frac{h^4 - h_f^4}{12}$	$\frac{h^4 - h_f^4}{6h}$
$\frac{1}{12} h^4 = \frac{1}{12} Ah^2$	$0.118h^3$
$\frac{h^4 - h_f^4}{12}$	$\frac{h^4 - h_f^4}{12h} \cdot \sqrt{2}$

No. of Section.	No. of Figure.	Form of Section.
VI.	8	
VII.	9	
VIII.	10	
IX.	11	
X.	12	

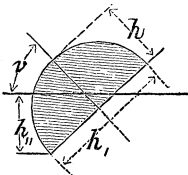
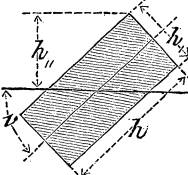
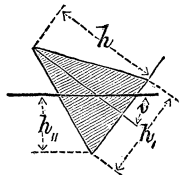
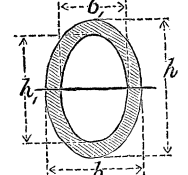
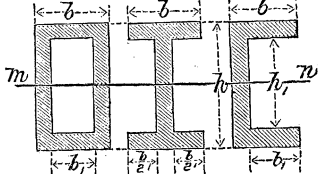
Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{48} bh^3$	$\frac{1}{24} bh^2$
$\frac{1}{48} (bh^3 - b, h,^3)$	$\frac{1}{24} \frac{bh^3 - b, h,^3}{h}$
$\frac{1}{48} bh^3 = \frac{1}{24} Ah^2$	$\frac{1}{24} bh^2$
$\frac{1}{36} gh^3 = \frac{1}{18} Ah^2$	$\frac{1}{24} gh^2$
$\frac{1}{48} bh^3 = \frac{1}{24} Ah^2$	$\frac{1}{24} bh^2$

No. of Section.	No. of Figure.	Form of Section.
XI.	13	
XII.	14	
XIII.	15	
XIV.	16	
XV.	17 and 18	

Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{4} \pi r^4 = \frac{1}{16} A d^2$	$\frac{1}{4} \pi r^3 = \frac{1}{4} A r$
$\frac{1}{4} \pi (r_1^4 - r_2^4)$	$\frac{1}{4} \pi \frac{r_1^4 - r_2^4}{r_1}$
$\frac{\pi}{64} d^4 - \frac{h^4}{12} = 0.0491 d^4 - \frac{h^4}{12}$	$\frac{I}{\frac{1}{2} d}$
$\frac{h^4}{12} - \frac{\pi}{64} d^4 = \frac{h^4}{12} - 0.0491 d^4$	$\frac{I}{\frac{1}{2} h}$
$\frac{12}{175} A h^2 = \frac{8}{175} b h^3$	$s = 0.576 h = (1 - \frac{4}{3\pi}) h$ $s_1 = 0.424 h = \frac{4}{3\pi} h$

No. of Section.	No. of Figure.	Form of Section.
XVI.	19	
XVII.	20	
XVIII.	21	
XIX.	22	
XX.	23	

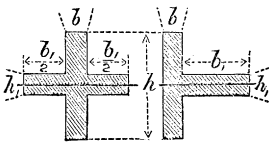
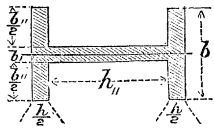
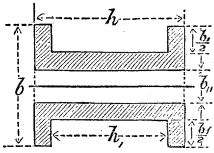
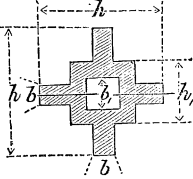
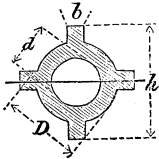
Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{30} bh^3 = \frac{1}{20} Ah^2$	$\frac{bh^2}{15} = \frac{1}{10} Ah$
$\frac{1}{64} \pi bh^3 = \frac{1}{16} Ah^2$	$\frac{1}{32} \pi bh^2 = \frac{1}{8} Ah$
$\frac{8}{175} bh^3 = \frac{12}{175} Ah^2$	$\frac{8}{105} bh^2 = \frac{4}{35} Ah$
$\frac{1}{30} bh^3 = \frac{1}{20} Ah^2$	$\frac{bh^2}{15} = \frac{1}{10} Ah$
$\frac{\pi}{64} bh^3 = \frac{1}{16} Ah^2$	$\frac{1}{32} \pi bh^2 = \frac{1}{8} Ah$

No. of Section.	No. of Figure.	Form of Section.
XXI.	24	
XXII.	25	
XXIII.	26	
XXIV.	27	
XXV.	28, 29, and 30	

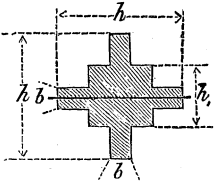
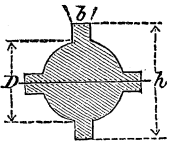
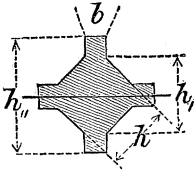

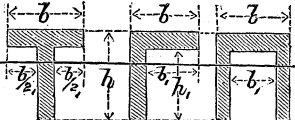
Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{5} A [\frac{1}{4} h,^2 \cos^2 v + \frac{1}{3} \frac{2}{5} h^2 \sin^2 v]$	$\frac{I}{h,,}$
$\frac{1}{12} A [h^2 \cos^2 v + h,^2 \sin^2 v]$	$\frac{I}{h,,}$
$\frac{1}{6} A [\frac{1}{4} h,^2 \cos^2 v + \frac{1}{3} h^2 \sin^2 v]$	$\frac{I}{h,,}$
$\frac{1}{64} \pi (bh^3 - b,h,^3)$	$\frac{I}{\frac{1}{2} h}$
$\frac{bh^3 - b,h,^3}{12}$	$\frac{bh^3 - b,h,^3}{6h}$

No. of Section.	No. of Figure.	Form of Section.
XXVI.	31	
XXVII.	32	
XVIII.	33	
XXIX.	34	
XXX.	35	

Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{bh^3 - b,h^3}{12}$	$\frac{bh^3 - b,h^3}{6h}$
$\frac{1}{12} [bh^3 - b,h^3 - (b-b,) h,,^3]$	$\frac{1}{6h} [bh^3 - b,h^3 - (b-b,) h,,^3]$
$\frac{1}{12} b [h^3 - h,,^3]$	$\frac{b (h^3 - h,,^3)}{6h}$
$\frac{1}{12} [bh^3 - bh,,^3 + b,h^3]$	$\frac{1}{6h} [bh^3 - bh,,^3 + b,h^3]$
$\frac{1}{12} [(bh^3 - b,h^3) - (b,,h^3)]$	$\frac{(bh^3 - b,h^3) - (b,,h^3)}{6h}$

No. of Section.	No. of Figure.	Form of Section.
XXXI.	36 and 37	
XXXII.	38	
XXXIII.	39	
XXXIV.	40	
XXXV.	41	

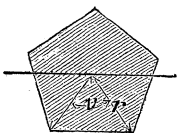
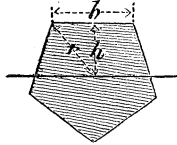
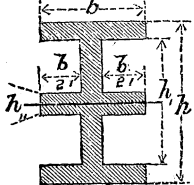
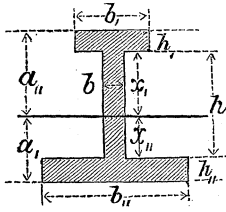
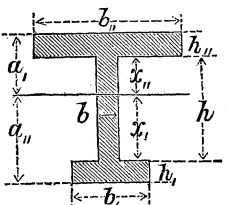
Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{1}{12} (bh^3 + b, h,^3)$	$\frac{bh^3 + b, h,^3}{6h}$
$\frac{1}{12} (hb^3 + h, b,^3)$	$\frac{hb^3 + h, b,^3}{6b}$
$\frac{1}{12} [(b^3h - 3b^2b, h, + 3bb,^2 h, - b,^3 h,) - (hb,^3)]$	$\frac{I}{\frac{1}{2} b}$
$\frac{1}{12} [h,^4 + b (h^3 - h,^3) + (h - h,) b^3 - b,^4]$	$\frac{I}{\frac{1}{2} h}$
$\frac{1}{12} [\frac{3}{16} \pi D^4 + b (h^3 - D^3) + (h - D) b^3] - 0.0491 d^4$	$\frac{I}{\frac{1}{2} h}$

No. of Section.	No. of Figure.	Form of Section.
XXXVI.	42	
XXXVII.	43	
XXXVIII.	44	
XXXIX.	45	
XL.	46, 47, and 48	

Moment of Inertia I	Moment of Resistance $\frac{I}{s}$
$\frac{1}{12} [h,^4 + b (h^3 - h,^3) + (h - h,) b^3]$	$\frac{I}{\frac{1}{2}h}$
$\frac{1}{12} [\frac{3}{16} \pi D^4 + b (h^3 - D^3) + (h - D) b^3]$	$\frac{I}{\frac{1}{2}h}$
$\frac{1}{12} [h,^4 + b (h^3 - h,^3) + (h - h,) b^3]$	$\frac{I}{\frac{1}{2}h,}$
$\frac{1}{12} [3 \pi (r,^4 - r,^4) + 2bh^3]$	$\frac{I}{\frac{1}{2}h,}$
$\frac{(bh^2 - b,h,^2)^2 - 4bh b,h, (h - h,)^2}{12 (bh - b, h,)}$	$\frac{(bh^2 - b,h,^2)^2 - 4bh b,h, (h - h,)^2}{6 (bh^2 - b,h,^2)}$

No. of Section.	No. of Figure.	Form of Section.
XLI.	49, 50, and 51	<p>The diagram shows three cross-sections of a material. The first is a solid rectangle with width b and height h. The second is a U-shaped section with outer width b, inner width b', and height h. The third is an L-shaped section with total width b and total height h. The flange width is $b/2$ and the web height is h'.</p>
XLII.	52	<p>A regular hexagon with a horizontal centerline. The radius from the center to a vertex is labeled r. The total height from the top flat face to the bottom flat face is labeled h.</p>
XLIH.	53	<p>A regular octagon with a horizontal centerline. The radius from the center to a vertex is labeled r. The total height from the top flat face to the bottom flat face is labeled h.</p>
XLIV.	54	<p>A regular octagon with a central hole. The radius from the center to a vertex of the outer octagon is labeled r. The total height from the top flat face to the bottom flat face is labeled h.</p>
XLV.	55	<p>A regular hexagon with a central hole. The radius from the center to a vertex of the outer hexagon is labeled r. The total height from the top flat face to the bottom flat face is labeled h.</p>

Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$\frac{(bh^2 - b_h h_s)^2 - 4bh b_h h_s (h - h_s)^2}{12 (bh - b_h h_s)}$	$\frac{(bh^2 - b_h h_s)^2 - 4bh b_h h_s (h - h_s)^2}{b (bh^2 + b_h h_s^2 - 2b_h h h_s)}$
$\frac{5}{16} r^4 \sqrt{3} = 0.5413 r^4$	$\frac{I}{\frac{1}{2}h}$
$\frac{1 + 2\sqrt{2}}{6} r^4 = 0.6381 r^4$	$\frac{I}{\frac{1}{2}h}$
$\frac{5}{16} \sqrt{3} (r^4 - r_s^4) = 0.5413 (r^4 - r_s^4)$	$\frac{I}{\frac{1}{2}h}$
$\frac{1 + 2\sqrt{2} (r^4 - r_s^4)}{6} = 0.6381 (r^4 - r_s^4)$	$\frac{I}{\frac{1}{2}h}$

No. of Section.	No. of Figure.	Form of Section.
XLVI.	56	
XLVII.	57	
XLVIII.	58	
XLIX.	59	
L.	60	

Moment of Inertia I .	Moment of Resistance $\frac{I}{s}$
$n_s = \text{number of sides.}$ $\frac{1}{24} n_s r^4 \sin.v (2 + \cos.v)$	$\frac{1}{24} n_s r^3 \sin.v (2 + \cos.v)$
$n_s = \text{number of sides.}$ $b = \text{length of a side.}$ $\frac{1}{12} A (3h^2 + \frac{1}{4} b^2)$	$\frac{1}{12} \frac{A}{h} (3h^2 + \frac{1}{4} b^2)$
$\frac{bh^3 - b_s h_s^3 + b_{ss} h_{ss}^3}{12}$	$\frac{bh^3 - b_s h_s^3 + b_{ss} h_{ss}^3}{6h}$
$I = \frac{1}{3} \left\{ \begin{array}{l} b_{ss} (a_s^3 - x_{ss}^3) + \\ b (x_{ss}^3 + x_s^3) + \\ b_s (a_{ss}^3 - x_s^3) \end{array} \right\};$ $x_s = \frac{bh^2 - b_s h_s^2 + b_{ss} h_{ss}^2 + 2b_{ss} h h_{ss}}{2(bh + b_s h_s + b_{ss} h_{ss})}$	$\frac{I}{a_s}$
$x_{ss} = h - x_s$ $a_s = x_{ss} + h_{ss}$ $a_{ss} = x_s + h_s$	$\frac{I}{a_{ss}}$

STRENGTH OF MATERIALS, &c.,

In lbs., avoirdupois, per square inch of cross-section.

Materials.	Weight of a cubic foot.	Ultimate Resistance to—				Modulus of elasticity.
		Tearing.	Crushing.	Shearing.	Cross-br'k. Modulus of Rupture.	
METALS.						
Brass, cast, average.....	505.7	18000	10300	9170000
“ wire.....	533	49000	14230000
Bronze or gun metal, (cop- per 8, tin 1)	524	36000	9900000
Copper, cast.....	537	19000	117000
“ sheet.....	549	30000
“ bolts.....	3 3000
“ wire.....	60000	17000000
Iron, cast, average.....	445	16500	112000	27700	17000000
“ various.....	434	13400	80000	14000000
.....	to	to	to	to
“ beams, average.....	456	29000	115000	28800	22900000
“ open work.....	17000
“ solid rect. bars, various qualities.	33000 to 43500
Iron, wrought, average.....	481	65000	36000 to 40000	50000
“ beams.....	38000
“ plates.....	51000
“ joints, d'ble riveted.	35700
Iron, wrought, joints, single riveted.	28600
Iron, wrought, bars and bolts.	60000 to 70000	29000000
“ hoop, best best.....	64000
“ wire.....	70000 to 100000	25300000
“ wire ropes.....	90000	15000000
Steel, average.....	490	100000	12000	15000	80000	29000000
“ bars.....	130000 to 80000	42000000
“ plates.....	80000
Lead, sheet.....	712	3300	7730	720000
Tin, cast.....	462	4600	15500	4000000
Zinc.....	436	7000 to 8000	13000000
TIMBER, (well seasoned and dry.)						
Ash.....	47	17000	9000	1400	12000 to 14000	1600000
Bamboo.....	25	6300
Beech.....	43	11500	9360	9000 to 20000	1350000

Materials.	Weight of a cubic foot.	Ultimate resistance to—				Modulus of elasticity.
		Tearing.	Crushing.	Shearing.	Cross-brk Modulus of Rupture.	
TIMBER—Continued.						
Birch.....	44	15000	6400	11700	1645000
Box.....	60	20000	10300
Chestnut.....	33.4	10000	5300	1066	1140000
		to				
Elm.....	34	13000	10300	1400	6000	700000
		14000			to	to
Ebony, West Indian.....	74.5	19000	9,000	1340000
Fir, Red Pine.....	37	12000	5375	500	2700	146 000
		to	to	to	7100	to
		14000	6200	800	9540	1900000
“ Spruce.....	37	12400	5900	600	9900	1400, 00
					to	to
					12300	1800000
“ Larch.....	33	9000	5570	970	50, 0	900000
		to		to	to	to
		10000		1700	10000	1360000
Hickory.....	52	25000	11000	1040000
Hornbeam.....	47	20000	7300
Lancewood.....	52.5	23400	17350
Locust.....	44	16000	9000	11, 00
Lignum vitæ.....	62	11800	9900	1200
Mahogany.....	35	8000	8200	10000	1255000
		to				
		21800				
Maple.....	49	10600	6500
Oak, British.....	52.5	10000	10000	2300	10000	1200000
		to			to	to
		19800			13000	1750000
“ Dantzic.....	47.4	7700	8700
“ American white.....	42	18000	6100
“ “ red.....	54	10250	6000	10609	2150000
Pine, American, white.....	34.6	11500	5300
“ “ yellow.....	29	15000	5400
Sycamore.....	37	13000	12000	9600
Teak, Indian.....	48	15000	12000	12000	2400000
					to	
					19000	
Water gum.....	62.5	11000	17460
Walnut.....	40	8000	6500
Willow, various.....	25	14000	4000	6600
Yew.....	50	8000
STONES, (natural and artificial.)						
Brick, weak red.....	125	550
			to			
			800			
“ strong red.....	135	to
“ fire.....	137.5	300	1100
“ work.....	100	1700
			417			
			to			
			612			
Cement.....	89	280
		to				
		300				

Materials.	Weight of a cubic foot.	Ultimate resistance to—				Modulus of elasticity.	
		Tearing.	Crushing.	Shearing.	Cross-br'k. Modulus of Rupture.		
STONES—Continued.							
Chalk.....	145.5	118	330			8000000	
Glass.....	173	9400		
Granite.....	168	5500 to 11000				
Limestone, marble.....	172	5500				
“ granular.....	197	4000 to 4500			13000000 to 16000000	
Mortar, hydraulic	100 to 170					
“ ordinary.....	109	50					
Rubble masonry.....	116	About 4-10 cut stone.				
Sandstone, strong.....	144	5500	2360		
“ ordinary.....		3300 to 4400				
“ weak.....	178	1100		
Slate.....		9600 to 12800	5000		
MISCELLANEOUS.							
Flaxen yarn.....	25000					
Hemp ropes.....	14000					
Hide, ox, undressed.....	6300					
Leather, ox.....	4200					
Silk fibre.....	5200					
Whalebone.....	7700					

MODULUS OF RUPTURE R .

According to Professor Rankine, the modulus of rupture is eighteen times the weight that is required to break a bar of a given material one inch square (section) and one foot between supports, the weight concentrated at the middle.

MODULUS OF ELASTICITY E

Is that power (in lbs. generally) through which a prismatic body of a given material, of section = 1, is assumed to be extended double its length, or compressed to 0.

Let A = Sectional area of a rod of the material.

W = Weight or power in lbs., which causes the extension or compression of the rod.

l = Length in inches of rod before W is applied.

γ = The extension or compression of the rod in inches, caused by W .

$$\text{Then will } E = \frac{Wl}{A\gamma}; \gamma = \frac{W}{AE} \cdot l.$$

FACTORS OF SAFETY k .

The ultimate resistance of material should be divided by—

Average, Steel and Wrought Iron.	For Proof Strength.	For Working Stress.
Steady load.....	2	3
Moving load.....	4 to 6
Cast Iron.		
Steady load.....	2 to 3	3 to 4
Moving load.....	6 to 8
Timber.		
Average	3	8 to 10

RESISTANCE TO CROSS-BREAKING AND SHEARING.

CAPACITY AND STRENGTH OF BEAMS.

Reference.

A = Area of cross-section of beam.

D = Deflection of beam from a horizontal.

E = Modulus of elasticity.

I = Moment of inertia of cross-section.

M = Maximum moment of rupture, or bending moment.

R = Modulus of rupture.

S = Vertical shearing force.

V = Pressure on supports.

W = Capacity or weight of load.

c, d, l = Dimensions in units of length.

k = Factor of safety.

w = Weight of load per unit of length.

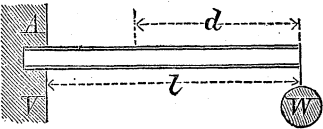
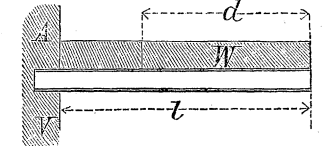
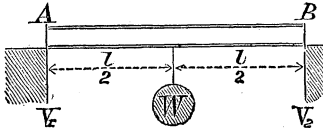
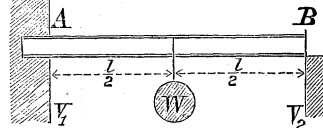
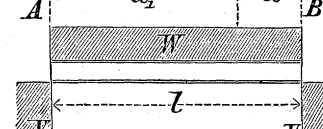
$\frac{I}{s}$ = Moment of resistance of cross-section.

For the stability of a beam: $M = K = \frac{R}{k} \cdot \frac{I}{s}$.

The web of a metal beam must have sufficient area to resist the shearing force S ; that is, $A = \frac{Sk}{\text{Ultimate resistance to shearing.}}$

The weight of the beam must be added to W , except in small beams, under 60 lbs. per lineal foot, when it may be disregarded.

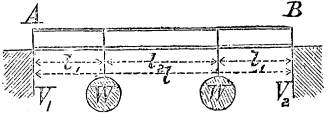
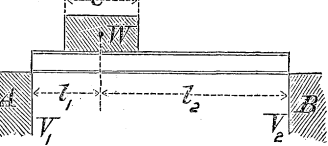
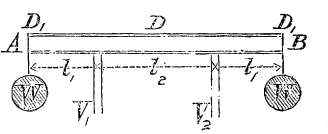
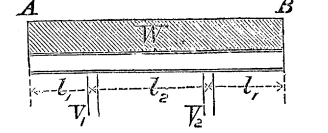
[NOTE.—Always use the same units of dimensions or weight.]

No. of Figure.	Manner of loading and fastening beams.	Maximum moment of rupture M .	Capacity W of any section.
61		$W \cdot l$	$\frac{K}{l}$
62		$W \cdot \frac{l}{2}$	$2 \frac{K}{l}$
63		$W \cdot \frac{l}{4}$	$4 \cdot \frac{K}{l}$
64		$W \cdot \frac{l}{5.333}$	$5.333 \frac{K}{l}$
65		$W \cdot \frac{l}{8}$	$8 \cdot \frac{K}{l}$

Maximum deflection D .	Distance from A to point of maximum D .	Shearing force S .	Pressure on supports V .
$\frac{W}{E.I} \cdot \frac{l^3}{3}$	l	At any point. W	W
$\frac{W}{E.I} \cdot \frac{l^3}{8}$	l	At any point. $w.d$	W
$\frac{W}{E.I} \cdot \frac{l^3}{48}$	$\frac{l}{2}$	At any point. $\frac{W}{2}$	$V_1 = V_2 = \frac{W}{2}$
$\frac{W.l^3}{E.I} \cdot 0.00931$	$0.553.l$	$\frac{3}{8} W \cdot \frac{l}{2}$	$V_1 = V_2 = \frac{W}{2}$
$\frac{5}{8} \frac{W}{E.I} \cdot \frac{l^3}{48}$	$\frac{l}{2}$	At any point, $d < d_1$; $w \left(\frac{l}{2} - d \right)$	$V_1 = V_2 = \frac{W}{2}$

No. of Figure.	Manner of loading and fastening beams.	Maximum moment of rupture M .	Capacity W of any section.
66		$W \cdot \frac{l}{8}$	$8 \cdot \frac{K}{l}$
67		$W \cdot \frac{l}{8}$	$8 \cdot \frac{K}{l}$
68		$W \cdot \frac{l}{12}$	$12 \cdot \frac{K}{l}$
69		$W \cdot l + W_1 \cdot l_1 + W_2 \cdot l_2$	
70		$W \cdot \frac{l_1 l_2}{l}$	$\frac{l}{l_1 l_2} \cdot K$

Maximum deflection D .	Distance from A to point of maximum D .	Shearing force S .	Pressure on supports V .
$\frac{W}{E.I} \cdot \frac{l^3}{4.48}$	$\frac{l}{2}$	$\frac{W}{2}$	$V_1 = V_2 = \frac{W}{2}$
$\frac{W.l^3}{E.I} \cdot 0.0054$	$0.572.l$	$w \cdot \left(\frac{3l}{8} - d \right)$	$V_1 = V_2 = \frac{W}{2}$
$\frac{W}{E.I} \cdot \frac{l^3}{8.48}$	$\frac{l}{2}$	$d < d_1$; $w \cdot \left(\frac{l}{2} - d \right)$	$V_1 = V_2 = \frac{W}{2}$
$\left(\frac{W_2}{E.I} \cdot \frac{l_2^3}{3} \right) +$ $\left(\frac{W_1}{E.I} \cdot \frac{l_1^3}{3} \right) +$ $\left(\frac{W}{E.I} \cdot \frac{l^3}{3} \right)$	l	At any point between loads. $S = W \cdot S_1 =$ $W + W_1 \cdot S_2 =$ $W + W_1 + W_2$	$W + W_1 + W_2$
$\frac{W}{E.I} \cdot \frac{l^3}{3} \cdot \frac{l_2^2}{l^2} \cdot \frac{l_1^2}{l^2}$		At any point and under any load. $S = W \cdot \frac{l_2}{l}$ Constant bet. A & W . $S_1 = W \cdot \frac{l_1}{l}$ Constant bet. B & W .	$V_1 = \frac{l_2}{l} W$ $V_2 = \frac{l_1}{l} W$

No. of Figure.	Manner of loading and fastening beams.	Maximum moment of rupture M .
71		$W.l_1$
72		$W \cdot \frac{l_1 l_2}{l} \left(1 - \frac{c}{2l}\right)$
73		$W.l_1$
74		<p>When $l > l_1 \sqrt{8}$;</p> $\frac{W}{2(l + 2l_1)} \cdot \left[\left(\frac{l}{2}\right)^2 - l_1^2 \right]$ <p>When $l < l_1 \sqrt{8}$;</p> $\frac{W l_1^2}{2(l + 2l_1)}$

Capacity W of any section.	Maximum deflection D .	Distance from A to point of maximum D .	Shearing force S .	Pressure on supports V .
$\frac{K}{l_1}$	$\frac{W}{E.I} \cdot \frac{l_2^3}{8} \cdot \frac{l_1}{l_2}$	$\frac{l}{2}$	W	$V_1 = V_2 = \frac{W}{2}$
$\frac{Kl}{l_1 l_2 \left(1 - \frac{c}{2l}\right)}$			$S \text{ at } A = W \frac{l_2}{l}$ $S \text{ at } B = W \frac{l_1}{l}$	$V_1 = \frac{l_2}{l} W$ $V_2 = \frac{l_1}{l} W$
$\frac{K}{l_1}$	$D = \frac{W l_2^2 l_1}{8 E.I}$ $D_1 = \frac{W l_1^2}{l} \cdot \left(\frac{l_2}{2} + \frac{l_1}{3}\right)$		W	$V_1 = V_2 = \frac{W}{2}$
$\frac{2(l + 2l_1)}{\left(\frac{l}{2}\right)^2 - l_1^2} K$			$w.l_1$ or $w \cdot \frac{l_2}{2}$	$V_1 = V_2 = \frac{W}{2}$
$\frac{2(l + 2l_1)}{l_1^2} K$			The greater value to be taken.	

No. of Figure.	Manner of loading and fastening beams.	Maximum moment of rupture M .
75		<p>When $d < (l - c)$; $W \cdot c \left(l - \frac{c}{2} - d \right)$</p> <p>When $d > (l - c)$; $\frac{W}{2} (l - d)^2$</p>
76		$W \frac{l_1^2 (3l - l_1) (l - l_1)}{2l^3}$
77		$W \frac{l_1^2 (l - l_1)}{l^2}$
78		$\left[W \frac{l_1}{l} (2l - l_1 - l_2) \right]$ $\left[W_1 \frac{l - l_2}{l} (l_1 + l_2) \right]$

Capacity W of any section.	Maximum deflection D .	Distance from A to point of maximum D .	Shearing force S .	Pressure on supports V .
$\frac{1}{c(l - \frac{c}{2} - d)} K$				
$\frac{2 K}{(l - d)^2}$				
$\frac{2l^3}{l_1^2(3l - l_1)(l - l_1)} K$	$\frac{W}{3 E \cdot I} \cdot \frac{l_1^2(l - l_1)}{l^2}$			
$\frac{l^2}{l_1^2(l - l_1)} K$				

EXAMPLE.—Capacity of wrought-iron **I**-shaped beams; top and bottom flange alike; load equally distributed; ends not fixed.

Dimensions of Cross-section.

h = Height = 10 inches.

b = Width of flange = 4 inches.

t = Thickness of flange = 0.8 inches.

t_w = Thickness of web = 0.5 inches.

$h_w = h - 2t$; $b_w = b - t_w$.

Distance between supports = 20 feet = 240 inches. Factor of safety = 3.

MOMENT OF RESISTANCE.

$$\frac{I}{s} = \frac{bh^3 - b_w h_w^3}{6h} = \frac{4 \times 10^3 - 3.5 \times 8.4^3}{6 \times 10} = 32.09.$$

Capacity W .

$$w = (4 \times 0.8 \times 2 + 8.4 \times 0.5) \times 240 \times 0.28 = 712.32 \text{ lbs.}$$

$$K = \frac{R}{k} \cdot \frac{I}{s} = \frac{38000}{3} \cdot 32.09 = 406473.33.$$

$$W = 8 \frac{K}{l} - w = 8 \cdot \frac{406473.33}{240} - 712.32 = 12836.72 \text{ lbs.}$$

EXAMPLE.—Capacity of cast-iron **I**-shaped beams; load equally distributed; ends not fixed; flange down.

Dimensions of Cross-section.

Let h = Height = 18 inches.

b = Width of flange = 9 inches.

t = Thickness of flange = 1.25 inches.

t_w = Thickness of web = 1 inch.

$h_w = h - t$; $b_w = b - t_w$.

Area = 28 square inches. Distance between supports = 20 feet = 240 inches. Factor of safety $k = 4$.

MOMENT OF RESISTANCE.

$$\begin{aligned} \frac{I}{s} &= \frac{1}{6} \left[\frac{(bh^2 - b_w h_w^2)^2}{bh^2 - 2b_w h_w + b_w^2} - \frac{4bh b_w (h - h_w)^2}{bh^2 - 2b_w h_w + b_w^2} \right] \\ &= \frac{1}{6} \left[\frac{(9 \times 18^2 - 8 \times 16.75^2)^2}{9 \times 18^2 - 2 \times 8 \times 18 \times 16.75 + 8 \times 16.75^2} - \right. \\ &\quad \left. \frac{4 \times 9 \times 18 \times 8 \times 16.75 (18 - 16.75)^2}{9 \times 18^2 - 2 \times 8 \times 18 \times 16.75 + 8 \times 16.75^2} \right] \end{aligned}$$

$$= \frac{1}{6} \left[\frac{452256.25}{336.5} - \frac{135675.00}{336.5} \right] = 157.$$

Capacity W.

$$w = 28 \times 240 \times 0.261 = 1754. \text{ lbs.}$$

$$K = \frac{R}{k} \cdot \frac{I}{s} = \frac{28000}{4} \cdot 157 = 1099000.$$

$$W = 8 \frac{K}{l} - w = 8 \cdot \frac{1000000}{240} - 1757 = 34879 \text{ lbs.}$$

For light beams no attention need be paid to weight of beam w .

CAPACITY W OF ROLLED I-SHAPED BEAMS.

Load equally distributed.

The calculations are based upon the patterns or sections used by the Phoenixville Iron Company. Practically this applies to all similar beams rolled in the United States, the difference in the profile of section being slight.

In the following table the factor of safety $k = 2.53$:

Reference.

W = Load in tons of 2,000 lbs., equally distributed.

w = Weight of beam in tons of 2,000 lbs.

L = Distance between supports in feet.

l = Distance between supports in inches.

w_1 = Weight per square foot of floor.

W_1 = Capacity of coupled or trebled beams in tons of 2,000 lbs.

D = Deflection in inches at centre, between supports.

d = Distance between centres of beams, when spacing for floors, in feet.

$$W = 8 \cdot \frac{K}{l} - w, K = \frac{R}{k} \cdot \frac{I}{s}, \frac{R}{k} = \frac{38000}{2.53} = 15000 \text{ lbs.} =$$

$$7.5 \text{ tons. } d = \frac{W}{L \cdot w_1}, \text{ or } d = \frac{W_1}{L \cdot w_1}, D = \frac{5}{8} \frac{W + w}{E \cdot I} \cdot \frac{l^3}{48}.$$

K^1 = Constant, computed by formulas. (See under examples.)

The rivets for coupled or trebled beams should be about $\frac{3}{4}$ inch in diameter, and 8 inches apart.

Trebled Beams.



Fig. 79.

$$W_1 = W \times 5.33.$$

Coupled Beams.



Fig 80.

$$W_1 = W \times 2.7.$$

Examples explanatory of the following Table.

EXAMPLE.—What is the capacity of a 15-inch light beam, load equally distributed, distance between supports = 20 feet?

$K^1 = \frac{K \cdot 8}{12}$, and $W = \frac{K^1}{L}$; for 15-inch light beam $\frac{K^1}{L} = \frac{345.19}{20} = 17.2 \text{ tons}$. This is also found at the intersection of 20 feet and column under capacity W .

EXAMPLE.—What distance apart should 9-inch medium beams be placed, the distance between supports being 20 feet, and to carry a total load of 140 lbs. per square foot of floor surface?

Ans. 4.4 feet; being found at the intersection of the horizontal line from 20 feet and the vertical column under 140 lbs.

EXAMPLE.—What is the capacity of 12-inch light beams trebled, load equally distributed, distance between supports = 25 feet?

Ans. W for 12-inch light beam = 9.19 and $W_1 = W \times 5.33 = 9.19 \times 5.33 = 48.98 \text{ tons}$.

CAPACITY OF ROLLED BEAMS.

Explanation of Tables for I Beams.

The first column gives the distance between supports in feet.

The second column gives the capacity in tons of 2,000 lbs., equally distributed.

The third column gives the deflection in inches at centre of beam.

The fourth column gives the weight of beam in lbs. for length between supports.

The fifth to fifteenth column (inclusive) gives the distance in feet that the beams should be spaced from centre to centre, for weight in lbs., per sq. ft. of surface for floors.

Pounds in decimals of a ton.

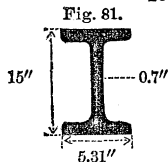
<i>lbs.</i>	<i>tons.</i>
60	= 0.03
70	= 0.035
80	= 0.04
90	= 0.045
100	= 0.05
140	= 0.07
160	= 0.08
180	= 0.085
200	= 0.1
250	= 0.125
300	= 0.15

In using these beams for floors, with brick arching, the ends resting on supports should have a bearing of about 8 inches, resting on a cast-iron plate, 8 × 12 in. sq., by 1 in. thick.

Tie rods should be used where floors are subject to heavy concentrated moving loads, (as trucks with merchandise, &c.;) these rods should be about 8 times the depth of beam apart, fastened about $\frac{1}{3}$ from the bottom of beam.

When beams are used to support walls, or as girders to carry floor beams, and put side by side (II;) they should be fastened together with cast-iron blocks, fitting between the flanges, so as to securely combine the two beams. The blocks may be put about the same distance apart as the tie-rods.

15' "Heavy" Beam. Weight per lf. = 66.66 lbs.



Sectional area..... = 20.0"

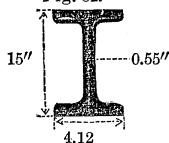
Moment of inertia I = 652.42Constant K' = 434.95

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—									
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.	250 lbs.
6	72.49	0.037	400.0										
7	62.13	0.050	466.6										
8	54.35	0.065	533.3										
9	48.32	0.084	600.0										
10	43.48	0.104	666.6										
11	39.54	0.126	733.3										
12	36.24	0.150	800.0										20.1
13	33.45	0.177	866.6										20.9
14	31.05	0.205	933.3										17.7
15	28.99	0.236	1000.0										15.5
16	27.18	0.270	1066.6										13.5
17	25.58	0.305	1133.3						21.4	19.6	17.0	15.0	12.0
18	24.16	0.342	1200.0						19.1	16.7	14.9	13.4	10.7
19	22.89	0.383	1266.6						17.6	15.2	13.4	12.0	9.6
20	21.73	0.426	1333.3					21.7	15.5	13.5	12.0	10.8	8.6
21	20.71	0.471	1400.0				22.0	19.7	14.7	12.5	11.5	9.8	7.9
22	19.58	0.515	1466.6				19.7	17.8	12.7	11.1	9.8	8.9	7.1
23	18.91	0.569	1533.3			21.0	18.9	17.1	11.8	10.5	9.4	8.2	6.7
24	18.12	0.623	1600.0		21.5	18.8	16.7	15.1	10.7	9.4	8.3	7.5	6.0
25	17.39	0.677	1666.6		19.9	17.3	15.5	14.4	10.2	8.6	7.7	6.9	5.6
26	16.72	0.735	1733.3	21.4	18.3	16.7	15.2	12.8	9.2	8.0	7.2	6.4	5.2
27	16.10	0.795	1800.0	19.8	17.1	14.9	13.4	11.9	8.5	7.4	6.7	5.9	4.7
28	15.53	0.860	1866.6	18.2	15.8	13.8	12.3	11.0	7.9	6.9	6.2	5.5	4.4
29	14.99	0.925	1933.3	17.2	14.8	12.9	11.5	10.7	7.4	6.8	5.7	5.1	4.1
30	14.49	0.994	2000.0	16.1	13.8	12.0	10.7	9.8	6.9	6.0	5.3	4.8	3.8
31	14.03	1.067	2066.6	15.0	12.9	11.3	10.0	9.0	6.4	5.6	5.0	4.5	3.6
32	13.59	1.141	2133.3	14.0	12.0	10.0	9.4	8.4	6.0	5.3	4.7	4.2	3.3
33	13.17	1.219	2200.0	13.3	11.4	9.9	8.8	7.9	5.7	4.9	4.4	3.9	3.1
34	12.79	1.304	2266.6	12.5	10.7	9.4	8.2	7.5	5.3	4.7	4.1	3.7	3.0
35	12.42	1.384	2333.3	11.8	10.1	8.8	7.9	7.1	5.0	4.4	3.9	3.5	2.8
36	12.08	1.473	2400.0	11.1	9.5	8.4	7.4	6.6	4.7	4.1	3.7	3.3	2.6
37	11.75	1.564	2466.6	10.8	9.1	7.9	7.0	6.3	4.4	3.9	3.5	3.1	2.5
38	11.43	1.656	2533.3	10.0	8.5	7.5	6.6	6.0	4.2	3.7	3.3	3.0	2.4
39	11.15	1.754	2600.0	9.5	8.1	7.1	6.3	5.7	4.0	3.5	3.1	2.8	2.2
40	10.87	1.854	2666.6	9.0	7.7	6.7	6.0	5.4	3.8	3.3	2.9	2.7	2.1

15'' "Light" Beam. Weight per lf. = 51.66 lbs.

Fig. 82.

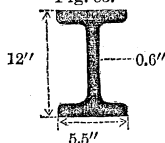


Sectional area..... = 15.5''
 Moment of inertia I = 517.78
 Constant K' = 345.19
 $W = \frac{K'}{L}$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—									
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.	250 lbs.
6	57.52	0.037	310.0										
7	49.31	0.050	361.6										
8	43.13	0.065	413.3										
9	38.35	0.084	465.0										
10	34.50	0.103	516.6	23.0
11	31.38	0.124	567.3	22.9
12	28.76	0.150	620.0	19.1
13	26.55	0.176	671.6	22.7	20.4	16.4	13.6
14	24.65	0.205	723.3	22.0	19.5	17.6	14.0	11.7
15	23.01	0.236	775.0	21.9	19.1	17.0	15.3	12.3	10.2
16	21.57	0.269	806.6	19.2	16.8	14.9	13.4	10.7	8.9
17	20.30	0.304	858.3	17.9	14.9	13.2	11.9	9.5	7.9
18	19.16	0.341	930.6	21.3	15.2	13.3	11.8	10.6	8.5	7.1
19	18.15	0.381	981.6	21.3	19.1	13.6	11.9	10.6	9.5	7.6
20	17.24	0.424	1033.3	21.5	19.1	17.2	12.3	10.7	9.5	8.6	6.9
21	16.43	0.469	1085.0	19.5	17.4	15.6	11.1	9.8	8.7	7.8	6.2
22	15.68	0.515	1136.6	20.3	17.8	15.8	14.2	10.1	8.9	7.9	7.1	5.7
23	15.00	0.565	1187.3	21.7	18.7	16.3	14.5	13.0	9.3	8.1	7.2	6.5	5.2
24	14.38	0.620	1240.0	19.9	17.1	14.9	13.3	11.9	8.5	7.4	6.6	5.9	4.8
25	13.80	0.674	1291.6	18.4	15.8	13.8	12.3	11.0	7.8	6.9	6.1	5.5	4.4
26	13.27	0.732	1343.3	17.0	14.5	12.8	11.3	10.2	7.2	6.3	5.6	5.1	4.0
27	12.78	0.791	1395.0	15.7	13.6	11.8	10.5	9.4	6.7	5.9	5.2	4.7	3.7
28	12.32	0.855	1446.6	14.6	12.5	11.0	9.7	8.8	6.2	5.5	4.8	4.4	3.5
29	11.93	0.921	1498.3	13.7	11.8	10.2	9.1	8.2	5.8	5.1	4.5	4.1	3.2
30	11.50	0.989	1550.0	12.7	10.9	9.5	8.5	7.6	5.4	4.7	4.2	3.8	3.0
31	11.13	1.060	1601.6	11.9	10.3	8.9	8.0	7.1	5.1	4.4	3.9	3.5	2.8
32	10.78	1.133	1653.3	11.2	9.6	8.4	7.4	6.7	4.8	4.2	3.7	3.3	2.6
33	10.46	1.211	1705.0	10.5	9.0	7.9	7.0	6.3	4.5	3.9	3.5	3.1	2.5
34	10.14	1.292	1756.6	9.9	8.5	7.4	6.6	5.9	4.2	3.7	3.3	2.9	2.3
35	9.86	1.375	1808.3	9.3	8.0	7.0	6.2	5.6	4.0	3.5	3.1	2.8	2.2
36	9.58	1.463	1860.0	8.8	7.6	6.6	5.9	5.3	3.8	3.3	2.9	2.6	2.1
37	9.32	1.553	1911.6	8.3	7.2	6.2	5.6	5.0	3.5	3.1	2.7	2.5	2.0
38	9.08	1.645	1963.3	7.9	6.8	5.9	5.3	4.7	3.4	2.9	2.6	2.3	1.9
39	8.85	1.742	2015.0	7.5	6.5	5.6	5.0	4.5	3.2	2.8	2.5	2.2	1.8
40	8.62	1.841	2066.6	7.1	6.1	5.3	4.7	4.3	3.0	2.6	2.3	2.1	1.7

12'' "Heavy" Beam. Weight per lf. = 56.66 lbs.

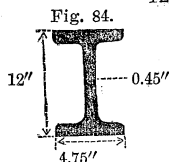
Fig. 83.



Sectional area..... = 17.0"
 Moment of inertia I = 373.53
 Constant K' = 311.23
 $W = \frac{K'}{L}$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	51.88	0.046	340.0									
7	44.54	0.063	396.6									
8	38.70	0.082	453.3									
9	34.58	0.105	510.0									
10	31.12	0.131	566.6	20.7
11	28.29	0.158	623.3	20.5
12	25.94	0.188	680.0	21.6	17.2
13	23.94	0.222	736.6	23.0	20.4	18.4	14.7
14	22.22	0.258	793.3	22.6	19.8	17.6	15.8	12.6
15	20.75	0.297	850.0	19.7	17.2	15.3	13.8	11.0
16	19.50	0.339	906.6	17.4	15.2	13.5	12.1	9.7
17	18.31	0.383	963.3	21.5	15.3	13.4	11.9	10.7	8.6
18	17.29	0.431	1020.0	21.3	19.2	13.7	12.0	10.6	9.6	7.6
19	16.38	0.481	1076.6	21.5	19.1	17.2	12.3	10.7	9.5	8.6
20	15.61	0.538	1133.3	19.5	17.9	15.6	11.1	9.7	8.6	7.8
21	14.82	0.592	1190.0	20.1	17.6	15.6	14.1	10.0	8.8	7.8	7.0
22	14.14	0.652	1246.6	21.4	18.3	16.0	14.2	12.8	9.1	8.0	7.1	6.4
23	13.53	0.717	1303.3	19.6	16.8	14.7	13.0	11.7	8.4	7.3	6.5	5.8
24	12.97	0.786	1360.0	18.0	15.4	13.5	12.0	10.8	7.7	6.7	6.0	5.4
25	12.45	0.855	1416.6	16.6	14.2	12.4	11.0	9.9	7.1	6.2	5.5	4.9
26	11.97	0.927	1473.3	15.3	13.1	11.5	10.2	9.2	6.5	5.7	5.1	4.6
27	11.52	1.003	1530.0	14.2	12.1	10.6	9.4	8.5	6.0	5.3	4.7	4.2
28	11.11	1.084	1586.6	13.2	11.3	9.9	8.8	7.9	5.0	4.9	4.4	3.9
29	10.73	1.170	1643.8	12.3	10.5	9.2	8.2	7.4	5.2	4.6	4.1	3.7
30	10.37	1.257	1700.0	11.5	9.8	8.6	7.6	6.9	4.9	4.3	3.8	3.4
31	10.04	1.350	1756.6	10.9	9.2	8.0	7.1	6.4	4.0	4.0	3.6	3.2
32	9.71	1.443	1813.3	10.1	8.6	7.5	6.7	6.0	4.3	3.7	3.4	3.0
33	9.43	1.546	1870.0	9.5	8.2	7.1	6.3	5.7	4.0	3.5	3.1	2.8
34	9.15	1.650	1926.6	8.9	7.6	6.7	5.9	5.3	3.8	3.3	2.9	2.6
35	8.89	1.758	1983.3	8.4	7.2	6.3	5.6	5.0	3.6	3.1	2.8	2.5
36	8.64	1.871	2040.0	8.0	6.8	6.0	5.3	4.8	3.4	3.0	2.6	2.4
37	8.41	1.987	2096.6	7.5	6.4	5.6	5.0	4.5	3.2	2.8	2.5	2.2
38	8.18	2.104	2153.3	7.1	6.1	5.3	4.7	4.3	3.0	2.6	2.3	2.1
39	7.98	2.234	2210.0	6.8	5.8	5.1	4.5	4.0	2.9	2.5	2.2	2.0
40	7.78	2.336	2266.6	6.4	5.5	4.8	4.3	3.8	2.7	2.4	2.1	1.9

12'' "Light" Beam. Weight per lf. = 41.66 lbs.

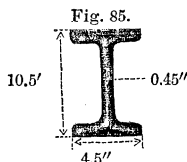


Sectional area..... = 12.5''

Moment of inertia I = 275.92Constant K' = 229.94

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in in.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	39.31	0.047	250.0									
7	32.84	0.063	291.6									
8	28.74	0.083	333.3									24.0
9	25.54	0.105	375.0									23.0
10	22.98	0.131	416.6									22.0
11	20.90	0.158	458.3							23.0	21.0	19.0
12	19.16	0.189	500.0						22.0	19.9	17.7	15.9
13	17.68	0.222	541.6						19.4	17.0	15.1	13.6
14	16.42	0.258	583.3						16.7	14.6	13.0	11.7
15	15.32	0.297	625.0				22.0	20.0	14.5	12.7	11.3	10.2
16	14.37	0.339	666.6			22.0	19.9	17.9	12.8	11.2	9.9	8.9
17	13.52	0.383	708.3			19.9	17.7	15.9	11.3	9.9	8.8	7.9
18	12.77	0.431	750.0		20.0	17.7	15.7	14.1	10.1	8.8	7.8	7.1
19	12.10	0.481	791.6	21.0	18.3	15.9	14.2	12.7	9.1	7.9	7.0	6.3
20	11.48	0.538	833.3	19.1	16.4	14.3	12.7	11.4	8.2	7.1	6.3	5.7
21	10.94	0.592	875.0	17.3	15.0	13.0	11.6	10.4	7.4	6.5	5.7	5.2
22	10.44	0.652	916.6	15.8	13.5	11.8	10.5	9.5	6.7	5.9	5.2	4.7
23	9.99	0.717	958.3	14.4	12.5	10.8	9.7	8.6	6.2	5.4	4.8	4.3
24	9.58	0.786	1000.0	13.3	11.4	9.9	8.8	7.9	5.7	4.9	4.4	3.9
25	9.19	0.855	1041.6	12.2	10.5	9.1	8.2	7.3	5.2	4.5	4.0	3.6
26	8.84	0.927	1083.3	11.3	9.7	8.5	7.5	6.8	4.8	4.2	3.7	3.4
27	8.51	1.003	1125.0	10.5	9.0	7.8	7.0	6.3	4.5	3.9	3.5	3.1
28	8.21	1.084	1166.6	9.7	8.3	7.3	6.5	5.8	4.1	3.6	3.2	2.9
29	7.92	1.170	1208.3	9.1	7.8	6.8	6.1	5.4	3.8	3.4	3.0	2.7
30	7.66	1.257	1250.0	8.5	7.2	6.3	5.6	5.1	3.6	3.1	2.8	2.5
31	7.41	1.350	1291.6	7.9	6.8	5.9	5.3	4.8	3.4	2.9	2.6	2.3
32	7.18	1.443	1333.3	7.4	6.4	5.6	4.9	4.4	3.2	2.8	2.4	2.2
33	6.96	1.542	1375.0	7.0	6.0	5.2	4.7	4.2	3.0	2.6	2.3	2.1
34	6.75	1.645	1416.6	6.6	5.6	4.9	4.4	3.9	2.8	2.4	2.2	2.0
35	6.57	1.754	1458.3	6.2	5.3	4.7	4.1	3.7	2.6	2.3	2.0	1.8
36	6.38	1.871	1500.0	5.9	5.0	4.4	3.9	3.5	2.5	2.2	1.9	1.7
37	6.21	1.987	1541.6	5.5	4.8	4.2	3.7	3.3	2.3	2.0	1.8	1.6
38	6.05	2.109	1583.3	5.3	4.5	3.9	3.5	3.1	2.2	1.9	1.7	1.5
39	5.89	2.229	1625.0	5.0	4.3	3.7	3.3	3.0	2.1	1.8	1.6	1.4
40	5.74	2.366	1666.6	4.7	4.1	3.5	3.1	2.8	2.0	1.7	1.5	1.3



10.5'' Beam. Weight per lf. = 35 lbs.

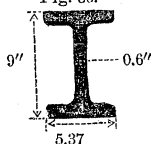
Sectional area..... = 10.5''
 Moment of inertia I = 179.44
 Constant K' = 170.903

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in in.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	23.48	0.053	210.0									
7	21.41	0.072	245.0									
8	21.36	0.095	280.0									
9	18.98	0.120	315.0									
10	17.09	0.149	350.0									
11	15.53	0.181	385.0									
12	14.24	0.216	420.0									
13	13.14	0.254	455.0									
14	12.20	0.295	490.0									
15	11.38	0.340	525.0									
16	10.68	0.389	560.0									
17	10.05	0.439	595.0									
18	9.49	0.494	630.0									
19	8.99	0.553	665.0									
20	8.54	0.614	700.0									
21	8.13	0.681	735.0									
22	7.75	0.752	770.0									
23	7.43	0.823	805.0									
24	7.12	0.903	840.0									
25	6.83	0.980	875.0									
26	6.57	1.067	910.0									
27	6.32	1.154	945.0									
28	6.10	1.251	980.0									
29	5.89	1.346	1015.0									
30	5.69	1.450	1050.0									
31	5.51	1.556	1085.0									
32	5.34	1.672	1120.0									
33	5.17	1.783	1155.0									
34	5.02	1.906	1190.0									
35	4.88	2.033	1225.0									
36	4.69	2.143	1260.0									
37	4.61	2.297	1295.0									
38	4.50	2.444	1330.0									
39	4.38	2.589	1365.0									
40	4.20	2.711	1400.0									

9" "Heavy" Beam. Weight per lf. = 50 lbs.

Fig. 86.



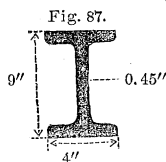
Sectional area..... = 15.0"

Moment of inertia I = 188.55Constant K' = 209.50

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—											
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.	250 lbs.	300 lbs.	
6	36.91	0.065	300.0												
7	29.92	0.084	350.0												
8	26.18	0.111	400.0	21.8
9	23.27	0.141	450.0	20.6	17.2
10	20.95	0.174	500.0	23.2	20.9	16.7	13.9	
11	19.04	0.211	550.0	21.6	19.1	17.3	13.7	11.5	
12	17.45	0.253	600.0	20.7	18.1	16.1	14.5	11.6	9.6	
13	16.11	0.297	650.0	17.6	15.4	13.8	12.3	9.9	8.2	
14	14.96	0.345	700.0	21.3	15.2	13.3	11.8	10.6	8.5	7.1	
15	13.96	0.398	750.0	20.6	18.6	13.2	11.6	10.3	9.3	7.5	6.2	
16	13.09	0.454	800.0	20.4	19.5	16.3	11.6	10.2	9.6	8.1	6.5	5.4	
17	12.32	0.515	850.0	20.7	18.1	16.1	14.5	10.3	9.0	8.0	7.2	5.7	4.8
18	11.63	0.580	900.0	21.5	18.4	16.1	14.3	12.9	9.2	8.0	7.1	6.4	5.1	4.3	
19	11.02	0.648	950.0	19.3	16.5	14.5	12.8	11.6	8.2	7.2	6.4	5.8	4.6	3.8	
20	10.47	0.722	1000.0	17.4	14.9	13.0	11.4	10.4	7.4	6.5	5.8	5.2	4.1	3.4	
21	9.97	0.799	1050.0	15.8	13.5	11.8	10.5	9.4	6.8	5.9	5.2	4.7	3.4	3.1	
22	9.52	0.883	1100.0	14.4	12.3	10.3	9.6	8.2	6.1	5.4	4.8	4.3	3.4	2.8	
23	9.10	0.968	1150.0	13.1	11.3	9.8	8.7	7.9	5.6	4.9	4.3	3.9	3.1	2.6	
24	8.72	1.062	1200.0	12.1	10.3	9.0	8.0	7.2	5.1	4.5	4.0	3.6	2.9	2.4	
25	8.34	1.152	1250.0	11.1	9.5	8.3	7.4	6.6	4.7	4.1	3.7	3.3	2.6	2.2	
26	8.05	1.258	1300.0	10.3	8.8	7.7	6.8	6.1	4.4	3.8	3.4	3.0	2.4	2.0	
27	7.75	1.363	1350.0	9.5	8.0	7.1	6.3	5.7	4.1	3.5	3.1	2.8	2.2	1.9	
28	7.48	1.477	1400.0	8.9	7.6	6.6	5.9	5.3	3.8	3.3	2.9	2.6	2.1	1.7	
29	7.22	1.593	1450.0	8.3	7.2	6.2	5.5	4.9	3.5	3.1	2.7	2.5	1.9	1.6	
30	6.98	1.718	1500.0	7.7	6.6	5.8	5.1	4.6	3.3	2.9	2.5	2.3	1.8	1.5	
31	6.75	1.846	1550.0	7.3	6.2	5.4	4.8	4.3	3.1	2.7	2.4	2.1	1.7	1.4	
32	6.54	1.982	1600.0	6.7	5.8	5.1	4.4	4.0	2.9	2.5	2.2	2.0	1.6	1.3	
33	6.34	2.119	1650.0	6.4	5.6	4.8	4.2	3.8	2.7	2.4	2.1	1.9	1.5	1.2	
34	6.16	2.265	1700.0	6.0	5.1	4.5	4.0	3.6	2.5	2.2	2.0	1.8	1.4	1.2	
35	5.98	2.416	1750.0	5.6	4.8	4.2	3.7	3.4	2.4	2.1	1.8	1.7	1.3	1.1	
36	5.81	2.577	1800.0	5.3	4.6	4.0	3.5	3.3	2.3	2.0	1.7	1.6	1.2	1.0	
37	5.66	2.742	1850.0	5.0	4.3	3.8	3.3	3.0	2.1	1.9	1.6	1.5	1.2		
38	5.51	2.918	1900.0	4.8	4.1	3.6	3.2	2.9	2.0	1.8	1.6	1.4	1.1		
39	5.37	3.098	1950.0	4.5	3.9	3.4	3.0	2.7	1.9	1.7	1.5	1.3	1.1		
40	5.27	3.289	2000.0	4.3	3.7	3.2	2.9	2.6	1.8	1.6	1.4	1.3	1.0		

9'' "Medium" Beam. Weight per lf. = 30 lbs.

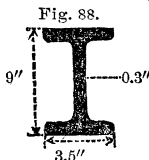


Sectional area..... = 9.0''
 Moment of inertia I = 111.32
 Constant K' = 123.69

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—									
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.	250 lbs.
6	20.60	0.062	180.0	22.0
7	17.67	0.085	210.0	25.0	20.0
8	15.46	0.111	240.0	24.0	21.0	19.0	15.0
9	13.74	0.141	270.0	21.0	19.0	16.0	15.0	11.0
10	12.36	0.174	300.0	17.0	15.0	13.0	12.0	9.8
11	11.24	0.211	330.0	22.0	20.0	14.0	12.0	11.0	10.0	8.1
12	10.30	0.252	360.0	21.0	19.0	17.0	12.0	10.0	9.5	8.5	6.8
13	9.51	0.297	390.0	29.0	18.0	16.0	14.0	10.0	9.1	8.1	7.3	5.8
14	8.83	0.345	420.0	21.0	18.0	15.0	14.0	12.0	9.0	7.8	7.0	6.3	5.0
15	8.24	0.398	450.0	18.0	15.0	13.0	12.0	10.0	7.8	6.8	6.1	5.4	4.3
16	7.73	0.455	480.0	16.0	13.0	12.0	10.0	9.6	6.9	6.0	5.3	4.8	3.8
17	7.21	0.511	510.0	14.0	12.0	10.0	9.4	8.4	6.0	5.3	4.7	4.2	3.2
18	6.87	0.580	540.0	12.0	10.0	9.5	8.4	7.6	5.4	4.7	4.2	3.8	3.0
19	6.51	0.650	570.0	11.0	9.7	8.5	7.6	6.8	4.8	4.2	3.8	3.4	2.7
20	6.18	0.722	600.0	10.0	8.8	7.7	6.8	6.1	4.4	3.8	3.4	3.0	2.4
21	5.88	0.799	630.0	9.3	8.0	7.0	6.2	5.6	4.0	3.5	3.1	2.8	2.2
22	5.62	0.884	660.0	8.5	7.2	6.3	5.6	5.1	3.6	3.1	2.8	2.5	2.0
23	5.37	0.969	690.0	7.7	6.6	5.8	5.1	4.6	3.3	2.9	2.5	2.3	1.8
24	5.15	1.065	720.0	7.1	6.1	5.3	4.7	4.2	3.0	2.6	2.3	2.1	1.7
25	4.94	1.157	750.0	6.5	5.6	4.9	4.3	3.9	2.8	2.5	2.1	1.9	1.5
26	4.83	1.277	780.0	6.1	5.3	4.6	4.1	3.7	2.6	2.3	2.0	1.8	1.4
27	4.58	1.365	810.0	5.6	4.8	4.2	3.7	3.3	2.4	2.1	1.8	1.6	1.3
28	4.41	1.476	840.0	5.2	4.5	3.9	3.5	3.1	2.2	1.9	1.7	1.5	1.2
29	4.26	1.593	870.0	4.8	4.1	3.6	3.2	2.9	2.0	1.8	1.6	1.4	1.1
30	4.12	1.718	900.0	4.5	3.9	3.4	3.0	2.7	1.9	1.7	1.5	1.3	1.0
31	3.99	1.846	930.0	4.2	3.6	3.2	2.8	2.5	1.8	1.6	1.4	1.2	1.0
32	3.86	1.982	960.0	4.0	3.4	3.0	2.6	2.4	1.7	1.5	1.3	1.2	
33	3.74	2.119	990.0	3.7	3.2	2.8	2.5	2.2	1.6	1.4	1.2	1.1	
34	3.63	2.265	1020.0	3.5	3.0	2.6	2.3	2.1	1.5	1.3	1.1	1.0	
35	3.53	2.416	1050.0	3.3	2.8	2.5	2.2	2.0	1.4	1.2	1.1	1.0	
36	3.43	2.577	1080.0	3.1	2.7	2.3	2.1	1.9	1.3	1.1	1.0		
37	3.34	2.742	1110.0	3.0	2.5	2.2	2.0	1.8	1.2	1.1	1.0		
38	3.25	2.918	1140.0	2.8	2.4	2.1	1.9	1.7	1.2	1.0			
39	3.17	3.098	1170.0	2.7	2.3	2.0	1.7	1.6	1.1	1.0			
40	3.09	3.289	1200.0	2.5	2.2	1.9	1.6	1.5	1.1				

9" "Light" Beam. Weight per lf. = 23.33 lbs.

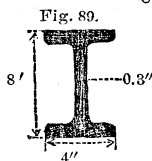


Sectional area..... = 7.0"
 Moment of inertia I = 91.06
 Constant K' = 101.2

$$W = \frac{K'}{L}.$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in pounds per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	16.86	0.062	140.0	22.0
7	14.45	0.085	163.3	25.0	22.0	20.6
8	12.65	0.111	186.6	25.0	19.7	17.5	15.8
9	11.24	0.141	210.0	17.8	15.6	13.8	12.5
10	10.12	0.175	233.3	22.0	20.0	14.4	12.6	11.2	10.1
11	9.20	0.212	256.6	21.0	18.7	16.7	11.9	10.4	9.2	8.3
12	8.43	0.253	280.0	23.0	20.0	17.5	15.6	14.0	10.0	8.7	7.8	7.0
13	7.78	0.297	303.3	19.9	17.9	14.9	13.4	11.9	8.5	7.4	6.6	5.9
14	7.22	0.345	326.6	17.1	14.7	12.8	11.4	10.3	7.3	6.4	5.7	5.1
15	6.74	0.398	350.0	14.9	12.9	11.2	10.0	8.9	6.4	5.6	5.0	4.5
16	6.31	0.453	373.3	13.1	11.2	9.8	8.7	7.8	5.6	4.9	4.3	3.9
17	5.95	0.510	396.6	11.6	10.0	8.7	7.8	7.0	5.0	4.3	3.8	3.5
18	5.62	0.579	420.0	10.4	8.9	7.8	6.9	6.2	4.4	3.9	3.4	3.1
19	5.32	0.648	443.3	9.3	8.0	7.0	6.2	5.6	4.0	3.5	3.1	2.8
20	5.06	0.721	466.6	8.4	7.2	6.3	5.6	5.0	3.6	3.1	2.8	2.5
21	4.81	0.797	490.0	7.6	6.5	5.7	5.1	4.5	3.2	2.8	2.5	2.2
22	4.59	0.879	513.3	6.9	5.9	5.2	4.6	4.1	2.9	2.6	2.3	2.1
23	4.40	0.968	536.6	6.3	5.5	4.7	4.2	3.8	2.7	2.3	2.1	1.9
24	4.21	1.060	560.0	5.8	5.0	4.3	3.8	3.5	2.5	2.1	1.9	1.7
25	4.04	1.151	583.3	5.3	4.6	4.0	3.5	3.2	2.3	2.0	1.7	1.6
26	3.89	1.254	606.6	4.9	4.2	3.7	3.3	2.9	2.1	1.8	1.6	1.4
27	3.74	1.359	630.0	4.6	3.9	3.4	3.0	2.7	1.9	1.7	1.5	1.3
28	3.60	1.466	653.3	4.2	3.6	3.2	2.8	2.5	1.8	1.6	1.4	1.2
29	3.48	1.582	676.6	4.0	3.4	3.0	2.6	2.4	1.7	1.5	1.3	1.1
30	3.37	1.711	700.0	3.7	3.2	2.8	2.5	2.2	1.6	1.4	1.2	1.1
31	3.26	1.837	723.3	3.5	3.0	2.6	2.3	2.1	1.5	1.3	1.1	1.0
32	3.16	1.968	746.6	3.2	2.8	2.4	2.1	1.9	1.4	1.2	1.0	
33	3.06	2.104	770.0	3.0	2.6	2.3	2.0	1.8	1.3	1.1		
34	2.97	2.250	793.3	2.9	2.4	2.1	1.9	1.7	1.2	1.0		
35	2.89	2.399	816.6	2.7	2.3	2.0	1.8	1.6	1.1			
36	2.81	2.565	840.0	2.6	2.2	1.9	1.7	1.5	1.0			
37	2.73	2.723	863.3	2.4	2.1	1.8	1.6	1.4				
38	2.66	2.898	886.6	2.3	2.0	1.7	1.5	1.4				
39	2.59	3.070	910.0	2.2	1.9	1.6	1.4	1.3				
40	2.52	3.250	933.3	2.1	1.8	1.5	1.4	1.2				

8'' Beam. Weight per lf. = 21.66 lbs.



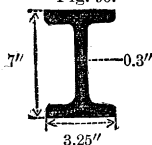
Sectional area..... = 6.5''
 Moment of inertia I = 65.99
 Constant K' = 82.49

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in pounds per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	13.74	0.070	130.0	22.9
7	11.78	0.095	151.6	21.0	18.6	16.8
8	10.30	0.124	173.3	25.7	18.3	16.0	14.3	12.8
9	9.16	0.158	195.0	20.3	14.5	12.7	11.3	10.1
10	8.23	0.198	216.6	20.5	18.3	16.4	11.7	10.2	9.1	8.2
11	7.49	0.238	238.3	22.6	19.4	17.0	15.1	13.6	9.7	8.5	7.5	6.8
12	6.87	0.284	260.0	19.0	16.3	14.3	12.7	11.4	8.1	7.1	6.3	5.7
13	6.34	0.335	281.6	16.2	14.0	12.1	10.9	9.7	6.9	6.0	5.4	4.8
14	5.89	0.390	303.3	14.0	12.0	10.5	9.3	8.4	6.0	5.2	4.6	4.2
15	5.49	0.447	325.0	12.2	10.5	9.1	8.1	7.3	5.2	4.5	4.0	3.6
16	5.15	0.511	346.6	10.7	9.1	8.0	7.1	6.4	4.5	4.0	3.6	3.2
17	4.85	0.580	368.3	9.5	8.0	7.1	6.3	5.7	4.0	3.7	3.2	2.8
18	4.58	0.653	390.0	8.4	7.2	6.3	5.6	5.1	3.8	3.1	2.7	2.5
19	4.34	0.731	411.6	7.6	6.5	5.7	5.1	4.5	3.3	2.9	2.5	2.2
20	4.11	0.810	433.3	6.8	5.8	5.1	4.5	4.1	2.9	2.5	2.2	2.0
21	3.92	0.898	455.0	6.2	5.3	4.6	4.1	3.7	2.6	2.3	2.0	1.8
22	3.73	0.989	476.6	5.6	4.8	4.2	3.7	3.4	2.4	2.1	1.8	1.6
23	3.58	1.090	498.3	5.1	4.4	3.8	3.4	3.1	2.2	1.9	1.7	1.5
24	3.42	1.192	520.0	4.7	4.0	3.5	3.1	2.8	2.0	1.7	1.5	1.4
25	3.29	1.300	541.6	4.3	3.7	3.2	2.9	2.6	1.8	1.5	1.4	1.3
26	3.17	1.417	563.3	4.0	3.4	3.0	2.7	2.4	1.7	1.4	1.3	1.2
27	3.05	1.536	585.0	3.7	3.2	2.8	2.5	2.2	1.6	1.4	1.2	1.1
28	2.94	1.662	606.6	3.5	3.0	2.6	2.3	2.1	1.5	1.3	1.1	1.0
29	2.84	1.795	628.3	3.2	2.8	2.4	2.1	1.9	1.4	1.2	1.0	
30	2.73	1.923	650.0	3.0	2.6	2.2	2.0	1.8	1.3	1.1		
31	2.66	2.080	671.6	2.8	2.4	2.1	1.9	1.7	1.2	1.0		
32	2.56	2.219	693.3	2.6	2.2	2.0	1.7	1.6	1.1			
33	2.49	2.383	715.0	2.5	2.1	1.8	1.6	1.4	1.0			
34	2.42	2.550	736.6	2.3	2.0	1.7	1.5	1.4				
35	2.35	2.722	758.3	2.2	1.9	1.6	1.4	1.3				
36	2.29	2.907	780.0	2.1	1.8	1.5	1.4	1.2				
37	2.22	3.084	801.6	2.0	1.7	1.5	1.3	1.2				
38	2.17	3.290	823.3	1.9	1.6	1.4	1.2	1.1				
39	2.11	3.484	845.0	1.8	1.5	1.3	1.2	1.0				
40	2.06	3.702	866.6	1.7	1.4	1.2	1.1	1.0				

7'' Beam. Weight per lf. = 18.33 lbs.

Fig. 90.



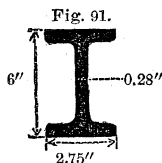
Sectional area..... = 5.5''

Moment of inertia I = 44.84Constant K' = 64.06

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—										
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.	250 lbs.	300 lbs.
6	10.67	0.080	110.0	25.4	22.2	19.7	17.7	14.2	11.8
7	9.15	0.109	128.3	18.6	16.3	14.5	13.0	10.5	8.7
8	8.00	0.143	146.6	25.0	22.2	20.0	14.2	12.5	11.1	10.0	8.0	6.6
9	7.11	0.181	165.0	22.9	19.7	17.5	15.8	11.2	9.8	8.7	7.9	6.3	5.2
10	6.40	0.224	183.3	21.3	18.2	16.0	14.2	12.8	9.1	8.0	7.1	6.4	5.1	4.2
11	5.82	0.272	201.6	17.6	15.3	13.2	11.7	10.5	7.5	6.6	5.8	5.2	4.2	3.5
12	5.33	0.325	220.0	14.8	12.6	11.1	9.8	8.8	6.3	5.5	4.9	4.4	3.5	2.9
13	4.92	0.382	238.3	12.6	10.9	9.4	8.3	7.5	5.4	4.7	4.1	3.7	3.0	2.5
14	4.56	0.444	256.6	10.8	9.3	8.1	7.2	6.5	4.6	4.0	3.6	3.2	2.6	2.1
15	4.27	0.513	275.0	9.4	8.2	7.1	6.3	5.7	4.0	3.5	3.1	2.8	2.2	1.8
16	3.99	0.585	293.3	8.3	7.1	6.2	5.5	4.9	3.5	3.1	2.7	2.4	1.9	1.6
17	3.76	0.665	311.6	7.3	6.5	5.5	4.9	4.4	3.1	2.7	2.3	2.1	1.7	1.4
18	3.55	0.749	330.0	6.5	5.6	4.9	4.3	3.9	2.8	2.4	2.1	1.9	1.5	1.3
19	3.37	0.840	348.3	5.9	5.1	4.4	3.9	3.5	2.5	2.2	1.9	1.7	1.4	1.1
20	3.20	0.936	366.6	5.3	4.5	4.0	3.5	3.2	2.2	2.0	1.7	1.6	1.2	1.0
21	3.05	1.038	385.0	4.8	4.1	3.6	3.2	2.9	2.0	1.8	1.6	1.4	1.1	
22	2.91	1.146	403.3	4.4	3.7	3.3	2.9	2.6	1.8	1.6	1.4	1.3	1.0	
23	2.78	1.257	421.6	4.0	3.4	3.0	2.7	2.4	1.7	1.5	1.3	1.2		
24	2.66	1.381	440.0	3.6	3.1	2.7	2.4	2.2	1.6	1.3	1.2	1.1		
25	2.56	1.504	458.3	3.4	2.9	2.5	2.2	2.0	1.5	1.2	1.1	1.0		
26	2.45	1.630	476.6	3.1	2.6	2.3	2.0	1.8	1.4	1.1				
27	2.37	1.775	495.0	2.9	2.5	2.1	1.9	1.7	1.3	1.0				
28	2.27	1.871	513.3	2.7	2.3	2.0	1.8	1.6	1.2					
29	2.20	2.075	531.6	2.5	2.1	1.8	1.7	1.5	1.1					
30	2.12	2.229	550.0	2.3	2.0	1.7	1.5	1.4	1.0					

6'' Beam. Weight per lf. = 13.33 lbs.



Sectional area..... = 4.0''

Moment of inertia I = 22.5

Constant K' = 37.64

$$W = \frac{K'}{L}$$

Dis. bet. supports in feet.	Capac. W in tons.	Deflec. in inches.	Weight in lbs.	Distance d bet. centres of beams in feet, for weight in lbs. per sq. foot of—								
				60 lbs.	70 lbs.	80 lbs.	90 lbs.	100 lbs.	140 lbs.	160 lbs.	180 lbs.	200 lbs.
6	6.27	0.094	80.0	23.2	20.9	14.9	13.0	11.6	10.4
7	5.37	0.128	93.3	19.1	17.3	15.3	10.0	9.5	8.4
8	4.70	0.168	106.6	19.5	16.8	14.6	13.0	11.7	8.5	7.3	6.5	5.8
9	4.18	0.213	120.0	15.4	13.4	11.6	10.4	9.2	6.6	5.8	5.1	4.6
10	3.75	0.263	133.3	12.5	10.7	9.3	8.3	7.5	5.3	4.7	4.1	3.7
11	3.42	0.320	146.6	10.3	9.0	7.7	6.9	6.2	4.4	3.8	3.4	3.1
12	3.13	0.382	160.0	8.6	7.0	6.5	5.7	5.2	3.7	3.2	2.9	2.6
13	2.89	0.450	173.3	7.4	6.4	5.5	4.9	4.4	3.1	2.7	2.4	2.2
14	2.68	0.524	186.6	6.3	5.4	4.7	4.2	3.8	2.7	2.3	2.1	1.9
15	2.51	0.607	200.0	5.5	4.8	4.2	3.7	3.3	2.3	2.1	1.8	1.6
16	2.34	0.689	213.3	4.8	4.1	3.6	3.2	2.9	2.0	1.8	1.6	1.4
17	2.21	0.786	226.6	4.3	3.7	3.2	2.9	2.5	1.8	1.6	1.4	1.3
18	2.09	0.888	240.0	3.8	3.3	2.9	2.5	2.3	1.6	1.4	1.2	1.1
19	1.98	0.995	253.3	3.4	3.0	2.6	2.3	2.1	1.4	1.3	1.1	
20	1.88	1.110	266.6	3.1	2.7	2.3	2.1	1.8	1.3	1.1		
21	1.79	1.231	280.0	2.8	2.4	2.1	1.9	1.7	1.2	1.0		
22	1.70	1.350	293.3	2.5	2.2	1.9	1.7	1.5	1.1			
23	1.63	1.493	306.6	2.3	2.0	1.7	1.5	1.4	1.0			
24	1.56	1.641	320.0	2.1	1.8	1.6	1.4	1.3				
25	1.50	1.787	333.3	2.0	1.7	1.5	1.3	1.2				
26	1.44	1.950	346.6	1.8	1.5	1.3	1.2	1.1				
27	1.39	2.129	360.0	1.7	1.4	1.2	1.1					
28	1.33	2.286	373.3	1.5	1.3	1.1						
29	1.29	2.489	386.6	1.4	1.2	1.0						
30	1.25	2.698	400.0	1.3	1.1							

CAST-IRON BEAMS.

Factor of rupture C for cast-iron beams of various sections.

The factor C is based on practical experiments by Hodgkinson. Its value alters with the different proportions of the cross-sections of beam.

Beam supported at the ends; load concentrated at the center.

Reference.

C = Factor of rupture.

W = Breaking weight in lbs.

A = Sectional area of beam in square inches.

l = Distance between supports in inches.

h = Height of beam in inches.

$$C = \frac{W \cdot l}{A \cdot h}, \quad W = \frac{A \cdot h}{l} \cdot C.$$

Dimensions in inches. b = Thickness of web at center is the unit.

Fig. 92.

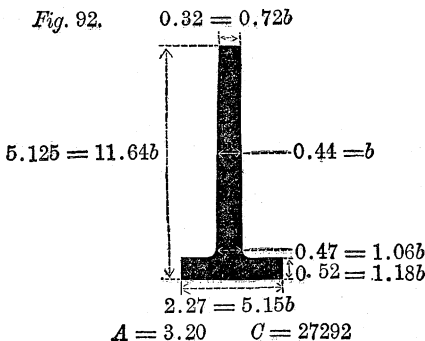


Fig. 93.

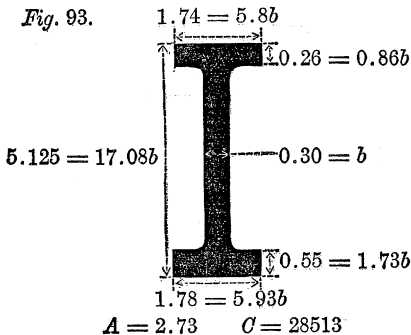


Fig. 94.

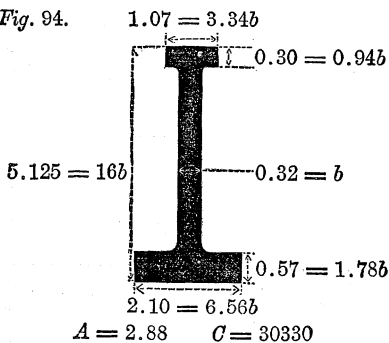


Fig. 95.

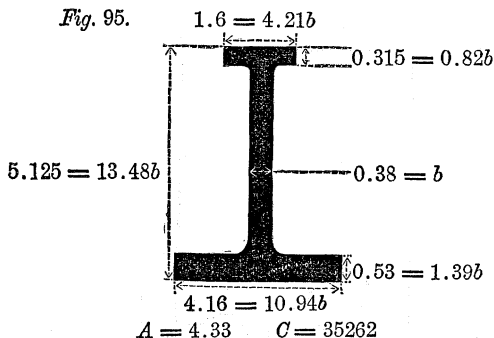
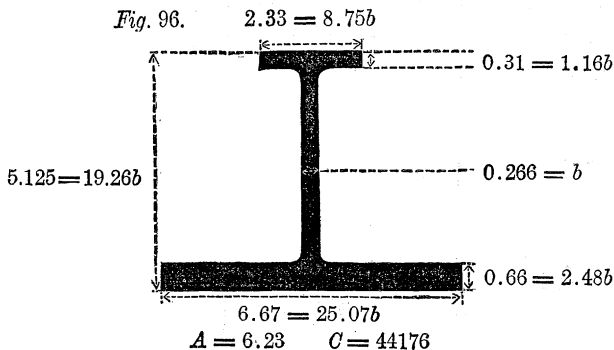
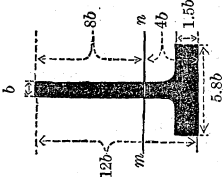
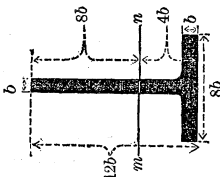


Fig. 96.

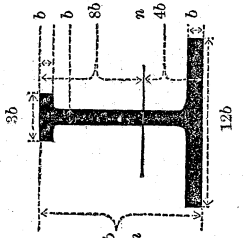
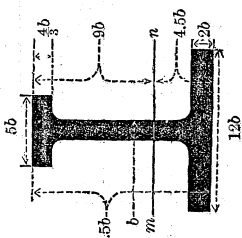


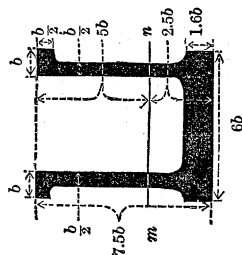
Theoretical cross-section of equal resistance, according to Moll and Reuleaux.

The following are theoretically the most economical sections. They form the basis for the table "Capacity of Cast-iron Beams." Flange nearest the neutral axis mn in tension:

No. of Figure.	Form of section.	Moment of inertia I .	Moment of resistance $\frac{I}{s}$	Sectional area A in inches.
97		$269.4b^4$	$33.675b^3$	$19.2b^2$
98		$278b^4$	$34.8b^3$	$19b^2$

Theoretical cross-section of equal resistance—Continued.

No. of Figure.	Form of section.	Moment of inertia I .	Moment of resistance $\frac{I}{s}$.	Sectional area A in inches.
99		$440b^4$	$55b^3$	$25b^2$
100		$922b^4$	$102.4b^3$	$40.82b^2$



101

Capacity of Cast-iron Beams of various Cross-sections in tons of 2,000 lbs. (Figs. 102 to 165.)

The following table gives the moment of resistance multiplied by the modulus of rupture:

The capacity W is equal to the tabulated coefficient K^1 divided by the distance l between supports, as shown in formulas below for variously circumstanced beams. For beams fixed at one end, l is distance between support and W at free end of beam.

The modulus of rupture is taken at $\frac{28000}{4} = 7,000$ lbs. at 4 times safety.

All dimensions in inches.

Capacity W in tons of 2,000 lbs.

Beam supported at both ends; principal flange at bottom.

Load equally distributed: $W = \frac{K^1}{\frac{1}{2}l}$, or $K^1 = \frac{l \cdot W}{2}$.

13.86²16.0756³80.2886⁴

Load concentrated at center: $W = \frac{K^1}{l}$, or $K^1 = l. W$.

Beam fixed at one end; principal flange at top.

Load equally distributed: $W = \frac{K^1}{2.l}$, or $K^1 = 2.l. W$.

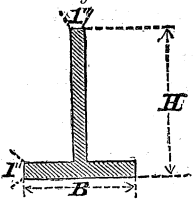
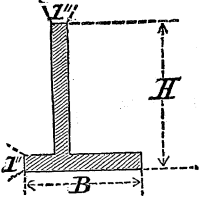
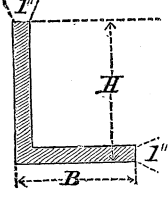
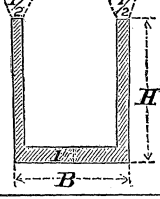
Load concentrated at free end: $W = \frac{K^1}{4.l}$, or $K^1 = 4.l. W$.

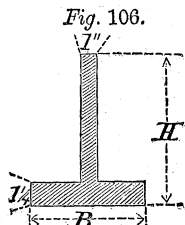
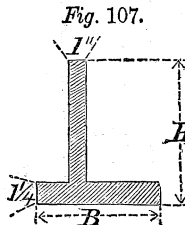
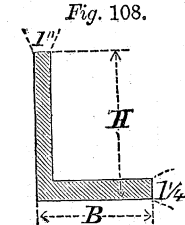
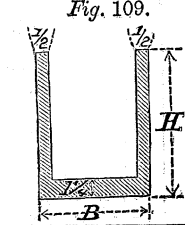

[NOTE.—The more the sectional area is contained in coefficient K^1 , the more is the section economical.]

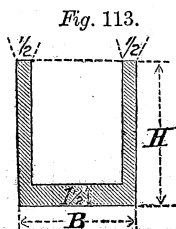
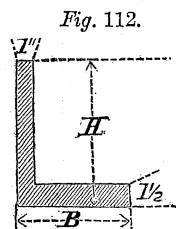
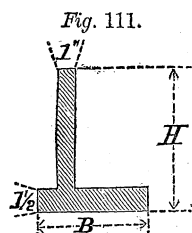
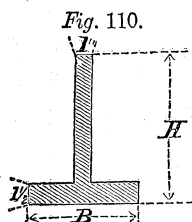
EXAMPLE.—Section No. 34. Load equally distributed; beam supported at both ends; thickness of web = 1 inch; thickness of flange = $1\frac{1}{4}$ inch; height = 10 inches; width of flange = 5.9 inches. Distance between supports = 20 feet = 240 inches.

$$W = \frac{K^1}{\frac{1}{2}l} = \frac{658}{120} = 5.48 \text{ tons capacity.}$$

The moment of resistance of cross-section $\frac{I}{8} = \frac{K^1}{14}$

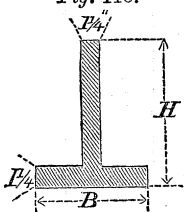
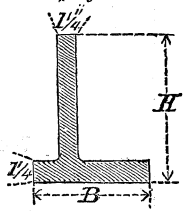
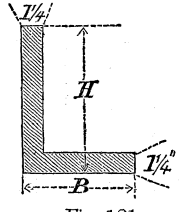
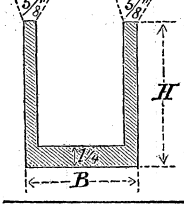
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1
<i>Fig. 102.</i> 	1	6	5.0	10.0	238
	2	6½	5.2	10.7	280
	3	7	5.5	11.5	322
	4	7½	5.7	12.2	364
	5	8	6.0	13.0	420
	6	8½	6.2	13.7	476
<i>Fig. 103.</i> 	7	9	6.5	14.5	532
	8	9½	6.7	15.2	602
	9	10	6.9	15.9	672
	10	10½	7.1	16.6	742
	11	11	7.4	17.4	812
	12	11½	7.6	18.1	882
	13	12	7.9	18.9	966
<i>Fig. 104.</i> 	14	12½	8.1	19.6	1050
	15	13	8.4	20.4	1134
	16	13½	8.6	21.1	1232
	17	14	8.8	21.8	1316
	18	14½	9.0	22.5	1428
	19	15	9.3	23.3	1526
<i>Fig. 105.</i> 	20	15½	9.5	24.0	1624
	21	16	9.8	24.8	1750
	22	16½	10.0	25.5	1848
	23	17	10.3	26.3	1960
	24	17½	10.5	27.0	2086
	25	18	10.8	27.8	2212

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1
<i>Fig. 106.</i> 	26	6	4.5	10.4	224
	27	6½	4.6	11.1	266
	28	7	4.8	11.8	322
	29	7½	5.0	12.5	364
<i>Fig. 107.</i> 	30	8	5.2	13.2	420
	31	8½	5.4	13.9	476
	32	9	5.6	14.7	532
	33	9½	5.7	15.4	588
<i>Fig. 108.</i> 	34	10	5.9	16.2	658
	35	10½	6.1	16.9	728
	36	11	6.3	17.6	798
	37	11½	6.5	18.3	882
<i>Fig. 109.</i> 	38	12	6.7	19.1	952
	39	12½	6.9	19.8	1036
	40	13	7.1	20.6	1134
	41	13½	7.3	21.3	1218
<i>Fig. 109.</i> 	42	14	7.5	22.1	1316
	43	14½	7.7	22.8	1414
	44	15	7.9	23.6	1512
	45	15½	8.0	24.3	1610
<i>Fig. 109.</i> 	46	16	8.2	25.1	1722
	47	16½	8.4	25.8	1834
	48	17	8.6	26.5	1946
	49	17½	8.8	27.2	2072
	50	18	9.0	28.0	2198

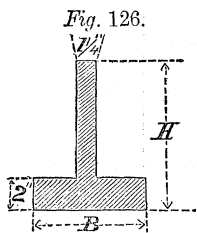
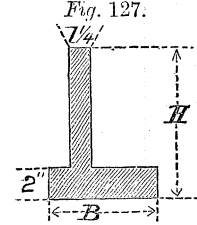
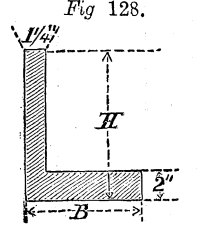
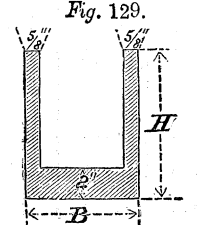


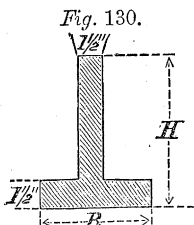
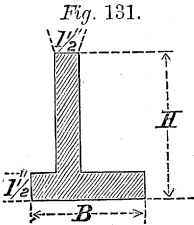
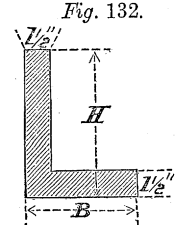
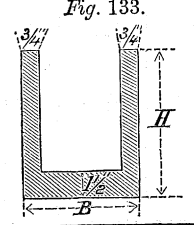
Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1
51	6	4.2	10.5	224
52	$6\frac{1}{2}$	4.3	11.4	266
53	7	4.5	12.3	308
54	$7\frac{1}{2}$	4.6	12.9	364
55	8	4.7	13.6	406
56	$8\frac{1}{2}$	4.8	14.3	462
57	9	5.0	15.0	532
58	$9\frac{1}{2}$	5.1	15.7	588
59	10	5.3	16.5	658
60	$10\frac{1}{2}$	5.4	17.2	728
61	11	5.6	17.9	798
62	$11\frac{1}{2}$	5.7	18.6	868
63	12	5.9	19.4	952
64	$12\frac{1}{2}$	6.0	20.1	1036
65	13	6.3	20.9	1120
66	$13\frac{1}{2}$	6.4	21.6	1204
67	14	6.6	22.4	1302
68	$14\frac{1}{2}$	6.7	23.1	1400
69	15	6.9	23.8	1498
70	$15\frac{1}{2}$	7.0	24.5	1610
71	16	7.2	25.3	1708
72	$16\frac{1}{2}$	7.3	26.0	1820
73	17	7.5	26.8	1932
74	$17\frac{1}{2}$	7.7	27.5	2058
75	18	7.9	28.3	2184

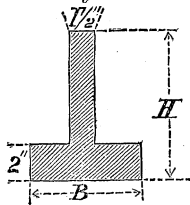
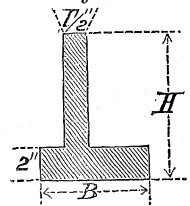
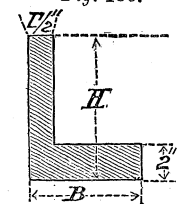
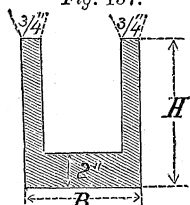
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1
<p><i>Fig. 114.</i></p>	76	6	4.0	12.0	224
	77	7	4.1	13.1	308
	78	8	4.2	14.4	406
<p><i>Fig. 115.</i></p>	79	9	4.4	15.7	518
	80	10	4.6	17.1	644
	81	11	4.8	18.6	784
<p><i>Fig. 116.</i></p>	82	12	5.0	20.0	938
	83	13	5.2	21.4	1106
	84	14	5.5	22.9	1288
<p><i>Fig. 117.</i></p>	85	15	5.7	24.4	1484
	86	16	5.9	25.8	1694
	87	17	6.2	27.3	1918
	88	18	6.4	28.8	2156

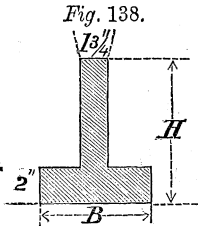
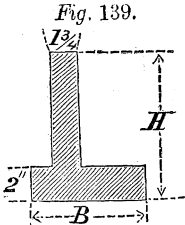
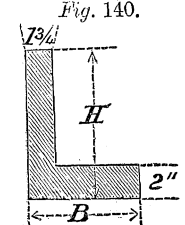
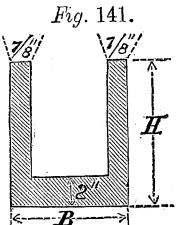
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 118.</i></p> 	89	6	5.6	12.9	294
	90	6½	5.8	13.8	336
	91	7	6.0	14.7	392
	92	7½	6.2	15.5	448
	93	8	6.4	16.4	518
<p><i>Fig. 119.</i></p> 	94	8½	6.6	17.3	588
	95	9	6.9	18.3	658
	96	9½	7.1	19.2	742
	97	10	7.4	20.2	826
	98	10½	7.6	21.1	910
<p><i>Fig. 120.</i></p> 	99	11	7.9	22.1	1008
	100	11½	8.1	23.0	1106
	101	12	8.4	23.9	1204
	102	12½	8.6	24.8	1302
	103	13	8.9	25.8	1414
<p><i>Fig. 121.</i></p> 	104	13½	9.1	26.7	1526
	105	14	9.4	27.7	1652
	106	14½	9.6	28.5	1764
	107	15	9.8	29.4	1890
	108	15½	10.0	30.3	2030
	109	16	10.3	31.3	2156
	110	16½	10.5	32.2	2296
	111	17	10.8	33.2	2436
	112	17½	11.0	34.1	2590
	113	18	11.3	35.0	2730

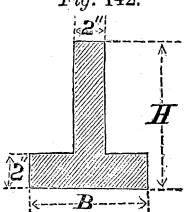
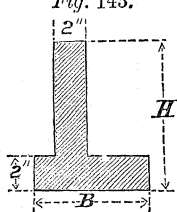
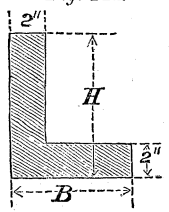
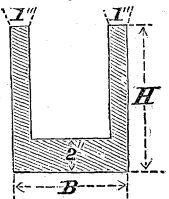
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1 .
Fig. 122.	114	6	5.3	13.6	280
	115	6½	5.4	14.4	336
	116	7	5.6	15.3	392
	117	7½	5.7	16.1	448
	118	8	5.9	17.0	518
Fig. 123.	119	8½	6.0	17.8	588
	120	9	6.2	18.7	658
	121	9½	6.4	19.6	742
	122	10	6.6	20.5	814
	123	10½	6.8	21.4	910
Fig. 124.	124	11	7.0	22.4	994
	125	11½	7.2	23.3	1092
	126	12	7.4	24.2	1190
	127	12½	7.6	25.1	1288
	128	13	7.8	26.1	1400
Fig. 125.	129	13½	8.0	27.0	1512
	130	14	8.2	27.9	1624
	131	14½	8.4	28.8	1750
	132	15	8.6	29.8	1876
	133	15½	8.8	30.7	2002
	134	16	9.0	31.6	2142
	135	16½	9.2	32.5	2282
	136	17	9.4	33.5	2422
	137	17½	9.6	34.4	2562
	138	18	9.8	35.3	2716

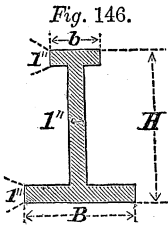
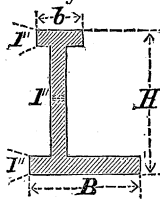
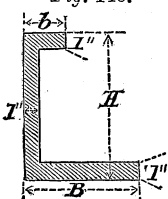
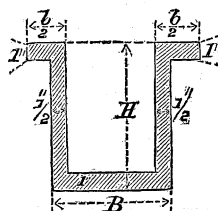
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 126.</i></p> 	139	6	5.0	15.0	280
	140	7	5.1	16.4	378
	141	8	5.3	18.0	504
<p><i>Fig. 127.</i></p> 	142	9	5.5	19.7	644
	143	10	5.7	21.4	798
	144	11	6.0	23.2	980
<p><i>Fig. 128.</i></p> 	145	12	6.3	25.0	1162
	146	13	6.5	26.8	1372
	147	14	6.8	28.6	1610
<p><i>Fig. 129.</i></p> 	148	15	7.1	30.5	1848
	149	16	7.4	32.3	2114
	150	17	7.7	34.2	2394
	151	18	8.0	36.0	2688

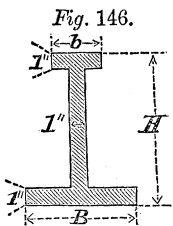
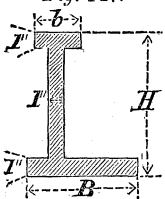
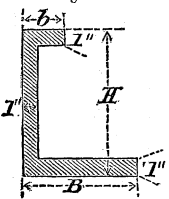
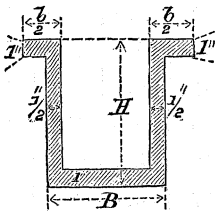
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K^1 .
 <p>Fig. 130.</p>	152	6	6.3	16.2	336
	153	6½	6.5	17.2	406
	154	7	6.7	18.3	476
	155	7½	6.9	19.3	546
	156	8	7.1	20.4	616
	157	8½	7.3	21.5	700
	158	9	7.5	22.6	784
 <p>Fig. 131.</p>	159	9½	7.7	23.6	882
	160	10	8.0	24.7	980
	161	10½	8.2	25.8	1078
	162	11	8.4	26.9	1190
	163	11½	8.6	28.0	1302
	164	12	8.9	29.1	1428
	165	12½	9.1	30.1	1554
 <p>Fig. 132.</p>	166	13	9.3	31.2	1680
	167	13½	9.5	32.3	1806
	168	14	9.8	33.5	1960
	169	14½	10.0	34.6	2100
	170	15	10.3	35.7	2254
	171	15½	10.5	36.8	2408
	172	16	10.8	38.0	2562
 <p>Fig. 133.</p>	173	16½	11.0	39.1	2730
	174	17	11.3	40.2	2912
	175	17½	11.5	41.3	3080
	176	18	11.8	42.5	3262

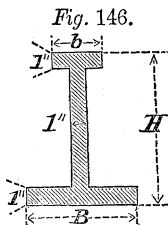
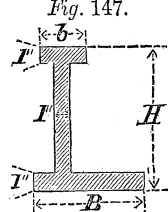
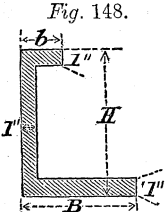
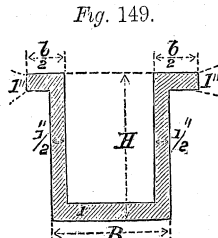
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<i>Fig. 134.</i>					
	177	6	6.0	18.0	336
	178	7	6.1	19.7	462
	179	8	6.3	21.6	602
<i>Fig. 135.</i>					
	180	9	6.6	23.6	770
	181	10	6.9	25.7	966
	182	11	7.2	27.9	1176
	183	12	7.5	30.0	1400
<i>Fig. 136.</i>					
	184	13	7.8	32.2	1652
	185	14	8.2	34.4	1932
	186	15	8.5	36.7	2212
<i>Fig. 137.</i>					
	187	16	8.9	38.8	2534
	188	17	9.2	41.0	2370
	189	18	9.6	43.2	3220

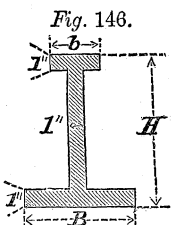
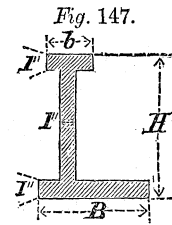
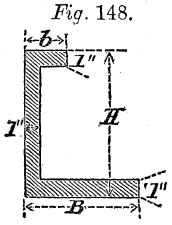
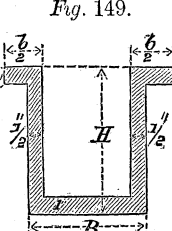
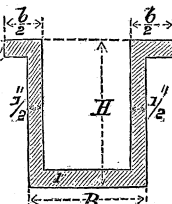
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 138.</i></p> 	190	6	7.0	21.0	392
	191	7	7.1	23.0	532
	192	8	7.4	25.2	714
<p><i>Fig. 139.</i></p> 	193	9	7.7	27.6	896
	194	10	8.0	30.0	1120
	195	11	8.4	32.5	1372
<p><i>Fig. 140.</i></p> 	196	12	8.8	35.0	1638
	197	13	9.1	37.5	1932
	198	14	9.6	40.1	2240
<p><i>Fig. 141.</i></p> 	199	15	10.0	42.7	2590
	200	16	10.4	45.2	2954
	201	17	10.8	47.8	3346
	202	18	11.2	50.4	3766

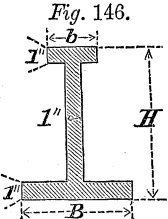
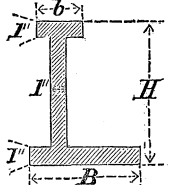
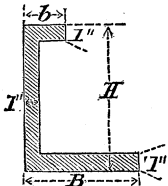
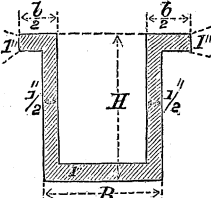
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Sectional area in square inches.	Coefficient K^1 .
<p><i>Fig. 142.</i></p> 	203	6	8.0	24.0	448
	204	7	8.1	26.2	616
	205	8	8.4	28.8	812
<p><i>Fig. 143.</i></p> 	206	9	8.8	31.5	1036
	207	10	9.1	34.3	1274
	208	11	9.6	37.1	1554
<p><i>Fig. 144.</i></p> 	209	12	10.0	40.0	1862
	210	13	10.4	42.9	2198
	211	14	10.9	45.8	2562
<p><i>Fig. 145.</i></p> 	212	15	11.4	48.7	2954
	213	16	11.8	51.7	3374
	214	17	12.3	54.6	3822
	215	18	12.8	57.6	4298

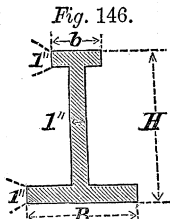
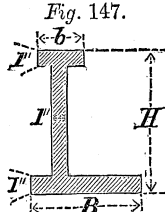
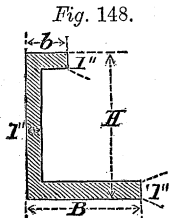
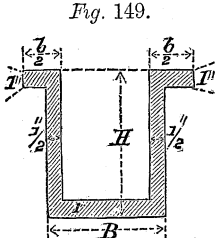
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 146.</i></p> 	1	6	6	1.4	11.4	294
	2	6	7	1.9	12.9	336
	3	6	8	2.3	14.3	392
	4	6	9	2.7	15.7	448
	5	6	10	3.1	17.1	504
	6	6	11	3.6	18.6	560
<p><i>Fig. 147.</i></p> 	7	6	12	4.0	20.0	602
	8	6	13	4.4	21.4	658
	9	6	14	4.9	22.9	714
	10	6	15	5.3	24.3	770
	11	6	16	5.7	25.7	826
	12	6	17	6.2	27.2	868
<p><i>Fig. 148.</i></p> 	13	6	18	6.6	28.6	924
	14	7	6	1.2	12.2	350
	15	7	7	1.7	13.7	420
	16	7	8	2.1	15.1	490
	17	7	9	2.6	16.6	560
	18	7	10	3.0	18.0	616
<p><i>Fig. 149.</i></p> 	19	7	11	3.4	19.4	686
	20	7	12	3.9	20.9	756
	21	7	13	4.3	22.3	826
	22	7	14	4.8	23.8	896
	23	7	15	5.2	25.2	966
	24	7	16	5.7	26.7	1022
	25	7	17	6.1	28.1	1092

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K^1 .
 <p>Fig. 146.</p>	26	7	18	6.5	29.5	1162
	27	8	6	1.0	13.0	434
	28	8	7	1.5	14.5	504
	29	8	8	1.9	15.9	588
	30	8	9	2.4	17.4	672
	31	8	10	2.8	18.8	742
 <p>Fig. 147.</p>	32	8	11	3.3	20.3	826
	33	8	12	3.7	21.7	910
	34	8	13	4.2	23.2	994
	35	8	14	4.6	24.6	1078
	36	8	15	5.1	26.1	1148
	37	8	16	5.5	27.5	1232
 <p>Fig. 148.</p>	38	8	17	6.0	29.0	1316
	39	8	18	6.4	30.4	1386
	40	9	7	1.3	15.3	588
	41	9	8	1.7	16.7	686
	42	9	9	2.2	18.2	784
	43	9	10	2.6	19.6	868
 <p>Fig. 149.</p>	44	9	11	3.1	21.1	966
	45	9	12	3.5	22.5	1064
	46	9	13	4.1	24.1	1162
	47	9	14	4.5	25.5	1246
	48	9	15	4.9	26.9	1344
	49	9	16	5.4	28.4	1442
	50	9	17	5.8	29.8	1526

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
 <p>Fig. 146.</p>	51	9	18	6.3	31.3	1624
	52	10	7	1.1	16.1	672
	53	10	8	1.5	17.5	784
	54	10	9	2.0	19.0	896
	55	10	10	2.4	20.4	1008
	56	10	11	2.9	21.9	1106
 <p>Fig. 147.</p>	57	10	12	3.3	23.3	1218
	58	10	13	3.8	24.8	1330
	59	10	14	4.3	26.3	1428
	60	10	15	4.7	27.7	1540
	61	10	16	5.2	29.2	1652
	62	10	17	5.7	30.7	1750
 <p>Fig. 148.</p>	63	10	18	6.1	32.1	1862
	64	11	8	1.3	18.3	896
	65	11	9	1.7	19.7	1008
	66	11	10	2.2	21.2	1134
	67	11	11	2.7	22.7	1246
	68	11	12	3.1	24.7	1372
 <p>Fig. 149.</p>	69	11	13	3.6	25.6	1498
	70	11	14	4.1	27.1	1610
	71	11	15	4.5	28.5	1736
	72	11	16	5.0	30.0	1862
	73	11	17	5.5	31.5	1974
	74	11	18	5.9	32.9	2100
	75	12	8	1.1	19.1	994

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 146.</p>	76	12	9	1.5	20.5	1120
	77	12	10	2.0	22.0	1260
	78	12	11	2.5	23.5	1400
	79	12	12	2.9	24.9	1526
	80	12	13	3.4	26.4	1666
 <p>Fig. 147.</p>	81	12	14	3.9	27.9	1806
	82	12	15	4.3	29.3	1932
	83	12	16	4.8	30.8	2072
	84	12	17	5.3	32.3	2198
	85	12	18	5.7	33.7	2338
 <p>Fig. 148.</p>	86	13	9	1.3	21.3	1232
	87	13	10	1.8	22.8	1386
	88	13	11	2.2	24.2	1540
	89	13	12	2.7	25.7	1680
	90	13	13	3.2	27.2	1834
 <p>Fig. 149.</p>	91	13	14	3.7	28.7	1988
	92	13	15	4.1	30.1	2128
	93	13	16	4.6	31.6	2282
	94	13	17	5.1	33.1	2422
	95	13	18	5.5	34.5	2576
 <p>Fig. 150.</p>	96	14	9	1.1	22.1	1358
	97	14	10	1.5	23.5	1512
	98	14	11	2.0	25.0	1680
	99	14	12	2.5	26.5	1834
	100	14	13	3.0	28.0	2002

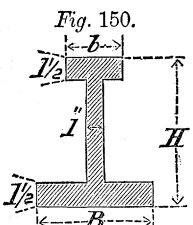
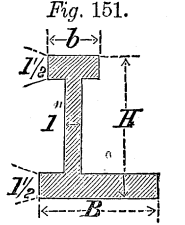
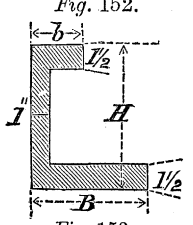
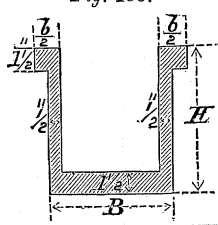
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 146.</i></p> 	101	14	14	3.4	29.4	2170
	102	14	15	3.9	30.9	2324
	103	14	16	4.4	32.4	2492
	104	14	17	4.8	33.8	2660
	105	14	18	5.3	35.3	2814
	106	15	10	1.3	24.3	1638
<p><i>Fig. 147.</i></p> 	107	15	11	1.8	25.8	1820
	108	15	12	2.3	27.3	2002
	109	15	13	2.7	28.7	2170
	110	15	14	3.2	30.2	2352
	111	15	15	3.7	31.7	2520
	112	15	16	4.2	33.2	2702
<p><i>Fig. 148.</i></p> 	113	15	17	4.6	34.6	2884
	114	15	18	5.1	36.1	3052
	115	16	10	1.1	25.1	1764
	116	16	11	1.6	26.6	1960
	117	16	12	2.0	28.0	2156
	118	16	13	2.5	29.5	2338
<p><i>Fig. 149.</i></p> 	119	16	14	3.0	31.0	2534
	120	16	15	3.5	32.5	2730
	121	16	16	3.9	33.9	2912
	122	16	17	4.4	35.4	3108
	123	16	18	4.9	36.9	3290
	124	17	11	1.3	27.3	2100
	125	17	12	1.8	28.8	2310

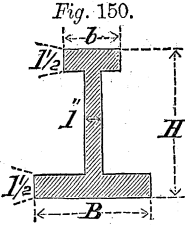
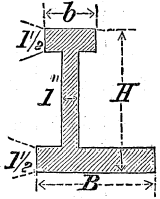
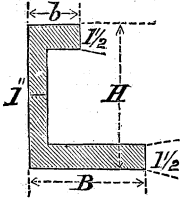
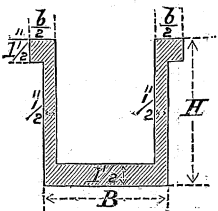
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 146.</p>	126	17	13	2.3	30.3	2506
	127	17	14	2.8	31.8	2716
	128	17	15	3.2	33.2	2926
 <p>Fig. 147.</p>	129	17	16	3.7	34.7	3122
	130	17	17	4.2	36.2	3332
	131	17	18	4.7	37.7	3542
 <p>Fig. 148.</p>	132	18	11	1.1	28.1	2240
	133	18	12	1.6	29.6	2464
	134	18	13	2.0	31.0	2688
 <p>Fig. 149.</p>	135	18	14	2.5	32.5	2898
	136	18	15	3.0	34.0	3122
	137	18	16	3.5	35.5	3346
	138	18	17	4.0	37.0	3556
	139	18	18	4.4	38.4	3780

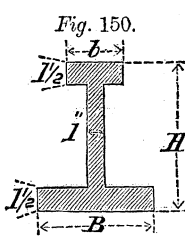
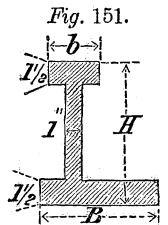
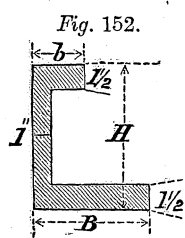
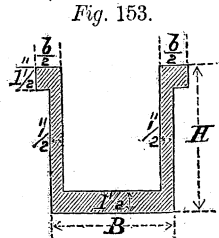
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 150.</i></p>	140	6	5	1.3	12.5	280
	141	6	6	1.7	14.6	336
	142	6	7	2.1	16.7	406
	143	6	8	2.5	18.8	462
	144	6	9	2.9	20.9	518
	145	6	10	3.2	22.8	588
	146	6	11	3.6	24.9	644
	147	6	12	4.0	27.0	714
	148	6	13	4.4	29.1	770
	149	6	14	4.8	31.2	826
<p><i>Fig. 151.</i></p>	150	6	15	5.2	33.3	896
	151	6	16	5.5	35.3	952
	152	6	17	5.9	37.4	1022
	153	6	18	6.3	39.5	1078
<p><i>Fig. 152.</i></p>	154	7	5	1.2	13.3	364
	155	7	6	1.6	15.4	434
	156	7	7	2.0	17.5	518
	157	7	8	2.4	19.6	602
	158	7	9	2.8	21.7	686
	159	7	10	3.2	23.8	756
	160	7	11	3.7	26.1	840
	161	7	12	4.1	28.2	924
	162	7	13	4.5	30.3	1008
	163	7	14	4.9	32.4	1092
<p><i>Fig. 153.</i></p>	164	7	15	5.3	34.5	1162

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
Fig. 150.	165	7	16	5.7	36.6	1246
	166	7	17	6.1	38.7	1330
	167	7	18	6.5	40.8	1414
	168	8	5	1.1	14.2	1022
	169	8	6	1.5	16.3	546
Fig. 151.	170	8	7	2.0	18.5	644
	171	8	8	2.4	20.6	742
	172	8	9	2.8	22.7	840
	173	8	10	3.2	24.8	938
	174	8	11	3.6	26.9	1036
Fig. 152.	175	8	12	4.1	29.2	1148
	176	8	13	4.5	31.3	1246
	177	8	14	4.9	33.4	1344
	178	8	15	5.3	35.5	1442
	179	8	16	5.7	37.6	1540
Fig. 153.	180	8	17	6.2	39.8	1638
	181	8	18	6.6	41.9	1750
	182	9	5	1.0	15.0	518
	183	9	6	1.4	17.1	644
	184	9	7	1.9	19.4	770
	185	9	8	2.3	21.5	882
	186	9	9	2.7	23.6	1008
	187	9	10	3.1	25.7	1120
	188	9	11	3.6	27.9	1246
	189	9	12	4.0	30.0	1358

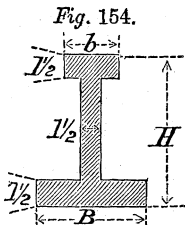
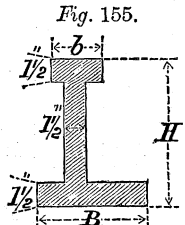
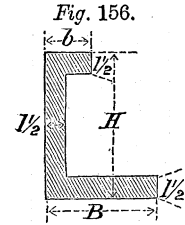
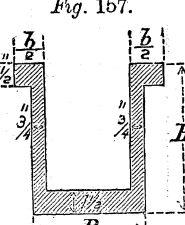
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 150.</i></p>	190	9	13	4.4	32.1	1484
	191	9	14	4.9	34.4	1610
	192	9	15	5.3	36.5	1722
	193	9	16	5.7	38.6	1848
	194	9	17	6.2	40.8	1960
	195	9	18	6.6	42.9	2086
<p><i>Fig. 151.</i></p>	196	10	6	1.3	18.0	756
	197	10	7	1.7	20.1	896
	198	10	8	2.2	22.3	1036
	199	10	9	2.6	24.4	1176
	200	10	10	3.1	26.7	1316
	201	10	11	3.5	28.8	1456
<p><i>Fig. 152.</i></p>	202	10	12	3.9	30.9	1596
	203	10	13	4.4	33.1	1736
	204	10	14	4.8	35.2	1876
	205	10	15	5.2	37.3	2016
	206	10	16	5.7	39.6	2156
	207	10	17	6.1	41.7	2296
<p><i>Fig. 153.</i></p>	208	10	18	6.5	43.8	2436
	209	11	6	1.2	18.8	854
	210	11	7	1.6	20.9	1022
	211	11	8	2.1	23.2	1176
	212	11	9	2.5	25.3	1344
	213	11	10	3.0	27.5	1498
	214	11	11	3.4	29.6	1666

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 150.</i></p> 	215	11	12	3.8	31.7	1820
	216	11	13	4.3	34.0	1974
	217	11	14	4.7	36.1	2128
	218	11	15	5.2	38.3	2296
	219	11	16	5.6	40.4	2464
	220	11	17	6.1	42.7	2618
	221	11	18	6.5	44.8	2786
<p><i>Fig. 151.</i></p> 	222	12	6	1.0	19.5	966
	223	12	7	1.5	21.8	1148
	224	12	8	1.9	23.9	1330
	225	12	9	2.4	26.1	1512
	226	12	10	2.8	28.2	1680
	227	12	11	3.3	30.5	1862
	228	12	12	3.7	32.6	2044
	229	12	13	4.2	34.8	2226
	230	12	14	4.6	36.9	2408
	231	12	15	5.1	39.2	2590
<p><i>Fig. 152.</i></p> 	232	12	16	5.5	41.3	2772
	233	12	17	6.0	43.5	2954
	234	12	18	6.4	45.6	3136
<p><i>Fig. 153.</i></p> 	235	13	7	1.4	22.6	1274
	236	13	8	1.8	24.7	1470
	237	13	9	2.3	27.0	1680
	238	13	10	2.7	29.1	1876
	239	13	11	3.2	31.3	2072

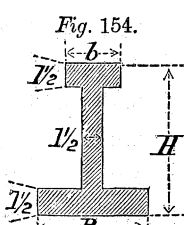
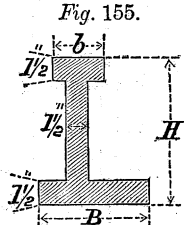
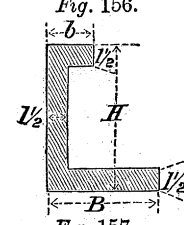
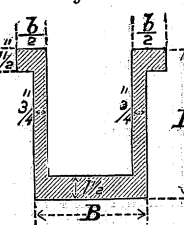
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 150.</p>	240	13	12	3.6	33.4	2282
	241	13	13	4.1	35.7	2478
	242	13	14	4.5	37.8	2674
	243	13	15	5.0	40.0	2884
	244	13	16	5.4	42.1	3080
	245	13	17	5.9	44.4	3276
	246	13	18	6.3	46.5	3486
 <p>Fig. 151.</p>	247	14	7	1.2	23.3	1400
	248	14	8	1.7	25.6	1624
	249	14	9	2.1	27.7	1848
	250	14	10	2.6	29.9	2058
	251	14	11	3.0	32.0	2282
	252	14	12	3.5	34.3	2506
	253	14	13	3.9	36.4	2730
 <p>Fig. 152.</p>	254	14	14	4.4	38.6	2954
	255	14	15	4.9	40.9	3178
	256	14	16	5.3	43.0	3388
	257	14	17	5.8	45.2	3612
	258	14	18	6.2	47.3	3836
	259	15	7	1.1	24.2	1526
	260	15	8	1.5	26.3	1764
 <p>Fig. 153.</p>	261	15	9	2.0	28.5	2016
	262	15	10	2.4	30.6	2254
	263	15	11	2.9	32.9	2492
	264	15	12	3.4	35.1	2744

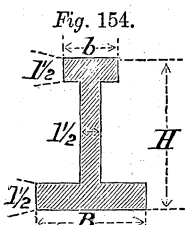
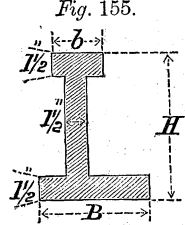
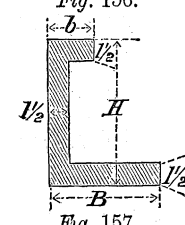
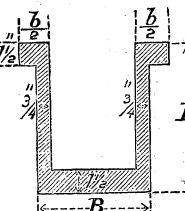
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 150.</p>	265	15	13	3.8	37.2	2982
	266	15	14	4.3	39.5	3220
	267	15	15	4.7	41.6	3472
	268	15	16	5.2	43.8	3710
	269	15	17	5.7	46.1	3948
	270	15	18	6.1	48.2	4200
 <p>Fig. 151.</p>	271	16	8	1.4	27.1	1918
	272	16	9	1.8	29.2	2184
	273	16	10	2.3	31.5	2450
	274	16	11	2.8	33.7	2702
	275	16	12	3.2	35.8	2968
	276	16	13	3.7	38.1	3234
 <p>Fig. 152.</p>	277	16	14	4.1	40.2	3500
	278	16	15	4.7	42.6	3766
	279	16	16	5.2	44.8	4018
	280	16	17	5.7	47.1	4284
	281	16	18	6.1	49.2	4550
	282	17	8	1.2	27.8	2072
 <p>Fig. 153.</p>	283	17	9	1.7	30.1	2352
	284	17	10	2.1	32.2	2632
	285	17	11	2.6	34.4	2926
	286	17	12	3.1	36.7	3206
	287	17	13	3.5	38.8	3486
	288	17	14	4.0	41.0	3766
	289	17	15	4.5	43.3	4060

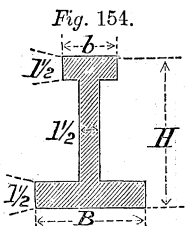
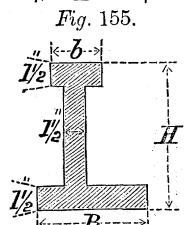
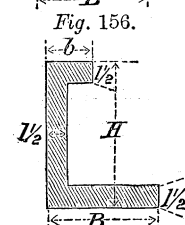
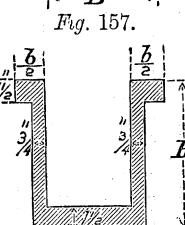
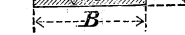
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p>Fig. 150.</p>	290	17	16	4.9	45.4	4340
	291	17	17	5.4	47.6	4620
	292	17	18	5.9	49.9	4900
<p>Fig. 151.</p>	293	18	8	1.1	28.7	2226
	294	18	9	1.5	30.8	2520
	295	18	10	2.0	33.0	2828
	296	18	11	2.5	35.3	3136
<p>Fig. 152.</p>	297	18	12	2.9	37.4	3430
	298	18	13	3.4	39.6	3738
	299	18	14	3.9	41.9	4056
<p>Fig. 153.</p>	300	18	15	4.3	44.0	4354
<p>Fig. 153.</p>	301	18	16	4.8	46.2	4648
	302	18	17	5.3	48.5	4956
	303	18	18	5.7	50.6	5269

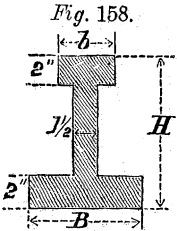
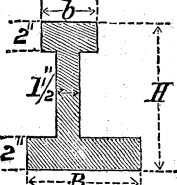
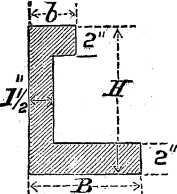
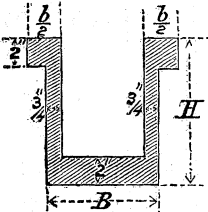
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 154.</p>	304	6	7	1.8	17.7	378
	305	6	8	2.2	19.8	448
	306	6	9	2.5	21.8	504
	307	6	10	2.9	23.9	574
	308	6	11	3.3	26.0	630
	309	6	12	3.7	28.1	686
 <p>Fig. 155.</p>	310	6	13	4.1	30.2	756
	311	6	14	4.5	32.3	812
	312	6	15	4.9	34.4	882
	313	6	16	5.2	36.3	938
	314	6	17	5.6	38.4	1008
	315	6	18	6.0	40.5	1064
 <p>Fig. 156.</p>	316	7	7	1.6	18.9	490
	317	7	8	2.0	21.0	574
	318	7	9	2.4	23.1	658
	319	7	10	2.8	25.2	742
	320	7	11	3.3	27.5	826
	321	7	12	3.7	29.6	896
 <p>Fig. 157.</p>	322	7	13	4.1	31.7	980
	323	7	14	4.5	33.8	1064
	324	7	15	4.9	35.9	1148
	325	7	16	5.3	38.0	1232
	326	7	17	5.7	40.1	1302
	327	7	18	6.1	42.2	1386
	328	8	8	1.9	22.4	714

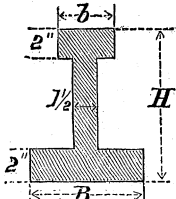
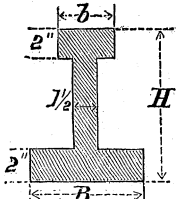
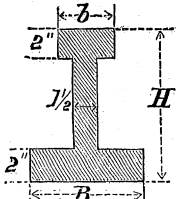
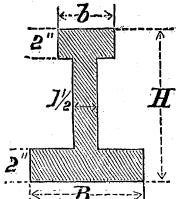
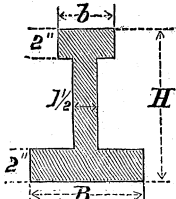
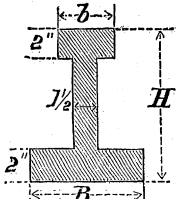
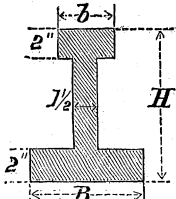
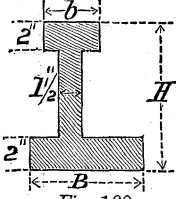
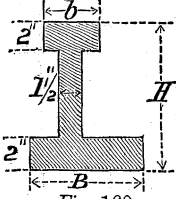
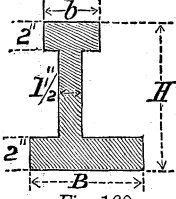
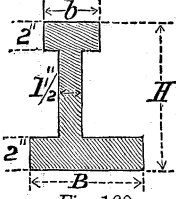
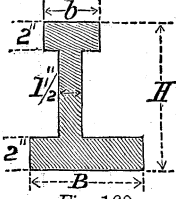
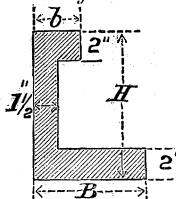
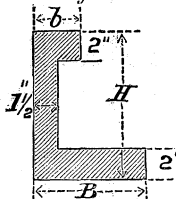
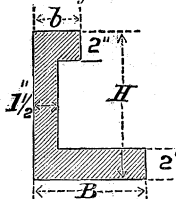
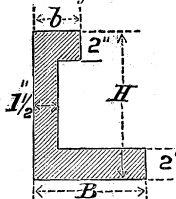
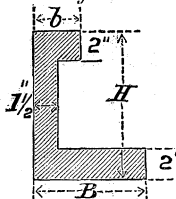
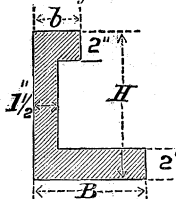
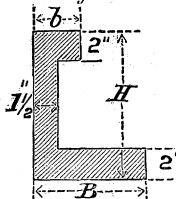
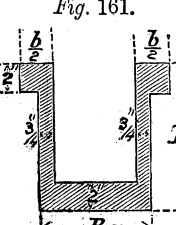
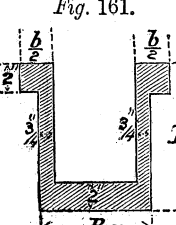
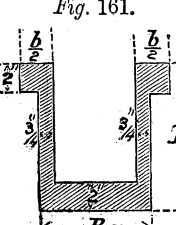
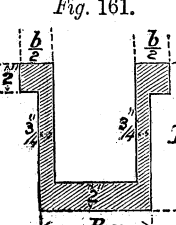
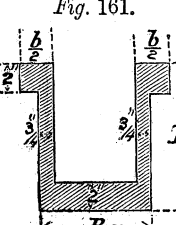
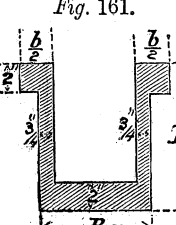
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
Fig. 154.	329	8	9	2.3	24.5	812
	330	8	10	2.7	26.6	910
	331	8	11	3.1	28.7	1008
	332	8	12	3.6	30.9	1106
	333	8	13	4.0	33.0	1218
	334	8	14	4.4	35.1	1316
Fig. 155.	335	8	15	4.8	37.2	1414
	336	8	16	5.2	39.3	1512
	337	8	17	5.7	41.6	1610
	338	8	18	6.1	43.7	1708
	339	9	8	1.7	23.6	840
	340	9	9	2.1	25.7	966
Fig. 156.	341	9	10	2.6	27.9	1092
	342	9	11	3.0	30.0	1204
	343	9	12	3.4	32.1	1330
	344	9	13	3.9	34.4	1442
	345	9	14	4.3	36.5	1568
	346	9	15	4.7	38.6	1694
Fig. 157.	347	9	16	5.1	40.7	1806
	348	9	17	5.6	42.9	1932
	349	9	18	6.0	45.0	2044
	350	10	8	1.5	24.8	980
	351	10	9	2.0	27.0	1120
	352	10	10	2.4	29.1	1260
	353	10	11	2.8	31.2	1400

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 154.</p>	354	10	12	3.3	33.5	1540
	355	10	13	3.7	35.6	1680
	356	10	14	4.1	37.7	1820
	357	10	15	4.6	39.9	1960
	358	10	16	5.0	42.0	2100
	359	10	17	5.5	44.3	2240
 <p>Fig. 155.</p>	360	10	18	5.9	46.4	2380
	361	11	9	1.8	28.2	1288
	362	11	10	2.2	30.3	1442
	363	11	11	2.6	32.4	1610
	364	11	12	3.1	34.7	1764
	365	11	13	3.5	36.8	1932
 <p>Fig. 156.</p>	366	11	14	4.0	39.0	2086
	367	11	15	4.4	41.1	2240
	368	11	16	4.9	43.4	2408
	369	11	17	5.3	45.5	2562
	370	11	18	5.8	47.7	2730
	371	12	9	1.6	29.4	1442
 <p>Fig. 157.</p>	372	12	10	2.0	31.5	1624
	373	12	11	2.5	33.8	1806
	374	12	12	2.9	35.9	1988
	375	12	13	3.4	38.1	2170
	376	12	14	3.8	40.2	2352
	377	12	15	4.2	42.3	2534
	378	12	16	4.7	44.6	2716

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 154.</p>	379	12	17	5.1	46.7	2898
	380	12	18	5.6	48.9	3066
	381	13	10	1.8	32.7	1806
	382	13	11	2.2	34.8	2002
	383	13	12	2.7	37.1	2212
	384	13	13	3.2	39.3	2408
 <p>Fig. 155.</p>	385	13	14	3.6	41.4	2618
	386	13	15	4.1	43.7	2814
	387	13	16	4.5	45.8	3010
	388	13	17	5.0	48.0	3220
	389	13	18	5.4	50.1	3416
	390	14	10	1.6	33.9	1988
 <p>Fig. 156.</p>	391	14	11	2.0	36.0	2212
	392	14	12	2.5	38.3	2436
	393	14	13	2.9	40.4	2660
	394	14	14	3.4	42.6	2870
	395	14	15	3.9	44.9	3094
	396	14	16	4.3	47.0	3318
 <p>Fig. 157.</p>	397	14	17	4.8	49.2	3542
	398	14	18	5.2	51.3	3766
	399	15	11	1.8	37.2	2408
	400	15	12	3.3	39.7	2660
	401	15	13	2.7	41.6	2898
	402	15	14	3.2	43.8	3136
	403	15	15	3.7	46.1	3388

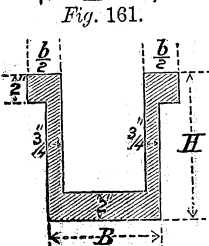
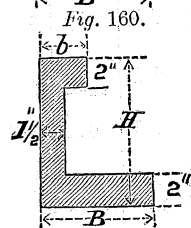
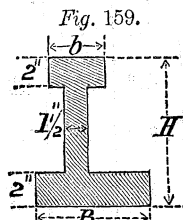
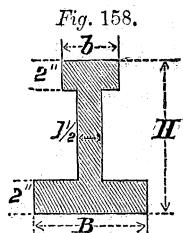
	Number of section.	Height H in inches.	Width B of lower flange in inches.	width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 154.</p>	404	15	16	4.1	48.2	3626
	405	15	17	4.6	50.4	3864
	406	15	18	5.0	52.5	4116
	407	16	11	1.6	38.4	2618
	408	16	12	2.1	40.7	2884
 <p>Fig. 155.</p>	409	16	13	2.5	42.8	3136
	410	16	14	3.0	45.0	3402
	411	16	15	3.4	47.1	3668
	412	16	16	3.9	49.4	3934
	413	16	17	4.4	51.6	4186
 <p>Fig. 156.</p>	414	16	18	4.8	53.7	4452
	415	17	12	1.8	41.7	3108
	416	17	13	2.3	44.0	3388
	417	17	14	2.8	46.2	3682
	418	17	15	3.2	48.3	3962
 <p>Fig. 157.</p>	419	17	16	3.7	50.6	4242
	420	17	17	4.2	52.8	4522
	421	17	18	4.6	54.9	4816
	422	18	12	1.6	42.9	3332
	423	18	13	2.1	45.2	3626
	424	18	14	2.5	47.3	3934
	425	18	15	3.0	49.5	4242
	426	18	16	3.5	51.8	4550
	427	18	17	3.9	53.9	4858
	428	18	18	4.4	56.1	5152

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 158.</i></p> 	429	6	6	1.5	18.0	336
	430	6	7	1.8	20.6	392
	431	6	8	2.2	23.4	462
	432	6	9	2.5	26.0	518
	433	6	10	2.8	28.6	588
	434	6	11	3.2	31.4	624
	435	6	12	3.5	34.0	714
<p><i>Fig. 159.</i></p> 	436	6	13	3.8	36.6	770
	437	6	14	4.2	39.4	840
	438	6	15	4.5	42.0	896
	439	6	16	4.8	44.6	952
	440	6	17	5.2	47.4	1022
	441	6	18	5.5	50.0	1078
<p><i>Fig. 160.</i></p> 	442	7	7	1.8	22.1	532
	443	7	8	2.2	24.9	616
	444	7	9	2.6	27.7	714
	445	7	10	2.9	30.3	798
	446	7	11	3.3	33.1	882
	447	7	12	3.7	35.9	966
<p><i>Fig. 161.</i></p> 	448	7	13	4.0	38.5	1050
	449	7	14	4.4	41.3	1134
	450	7	15	4.7	43.9	1218
	451	7	16	5.1	46.7	1302
	452	7	17	5.5	49.5	1386
	453	7	18	5.8	52.1	1470

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
<i>Fig. 158.</i>						
	454	8	7	1.8	23.6	686
	455	8	8	2.2	26.4	714
	456	8	9	2.5	29.0	896
	457	8	10	2.9	31.8	1008
	458	8	11	3.3	34.6	1120
	459	8	12	3.7	37.4	1232
	460	8	13	4.1	40.2	1344
<i>Fig. 159.</i>						
	461	8	14	4.5	43.0	1456
	462	8	15	4.9	45.8	1554
	463	8	16	5.2	48.4	1666
	464	8	17	5.6	51.2	1778
	465	8	18	6.0	54.0	1890
<i>Fig. 160.</i>						
	466	9	7	1.7	24.9	826
	467	9	8	2.1	27.7	966
	468	9	9	2.5	30.5	1106
	469	9	10	2.9	33.3	1232
	470	9	11	3.3	36.1	1372
	471	9	12	3.7	38.9	1498
	472	9	13	4.1	41.7	1638
<i>Fig. 161.</i>						
	473	9	14	4.5	44.5	1778
	474	9	15	4.9	47.3	1904
	475	9	16	5.3	50.1	2044
	476	9	17	5.7	52.9	2184
	477	9	18	6.1	55.7	2310
	478	10	7	1.6	26.2	980

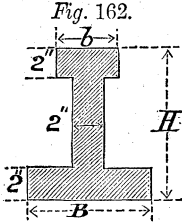
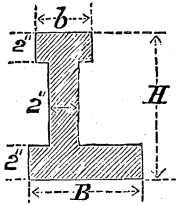
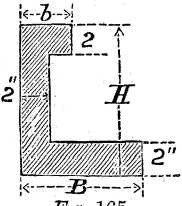
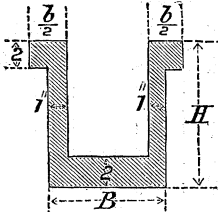
	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
<p><i>Fig. 158.</i></p>	479	10	8	2.0	29.0	1134
	480	10	9	2.4	31.8	1302
	481	10	10	2.8	34.6	1456
	482	10	11	3.2	37.4	1624
	483	10	12	3.6	40.2	1778
	484	10	13	4.0	43.0	1946
	485	10	14	4.4	45.8	2100
<p><i>Fig. 159.</i></p>	486	10	15	4.9	48.8	2268
	487	10	16	5.3	51.6	2422
	488	10	17	5.7	54.4	2590
	489	10	18	6.1	57.2	2744
	490	11	8	1.9	30.3	1316
	491	11	9	2.3	33.1	1512
	492	11	10	2.7	35.9	1694
<p><i>Fig. 160.</i></p>	493	11	11	3.1	38.7	1876
	494	11	12	3.5	41.5	2072
	495	11	13	4.0	44.5	2254
	496	11	14	4.4	47.3	2436
	497	11	15	4.8	50.1	2632
	498	11	16	5.2	52.9	2814
	499	11	17	5.6	55.7	2996
<p><i>Fig. 161.</i></p>	500	11	18	6.1	58.7	3192
	501	12	8	1.7	31.4	1512
	502	12	9	2.1	34.2	1722
	503	12	10	2.6	37.2	1932

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
Fig. 158.	504	12	11	3.0	40.0	2142
	505	12	12	3.4	42.8	2366
	506	12	13	3.9	45.8	2576
	507	12	14	4.3	48.6	2786
	508	12	15	4.7	51.4	2996
Fig. 159.	509	12	16	5.2	54.4	3220
	510	12	17	5.6	57.2	3430
	511	12	18	6.0	60.0	3640
	512	13	8	1.6	32.7	1680
	513	13	9	2.0	35.5	1932
Fig. 160.	514	13	10	2.4	38.3	2170
	515	13	11	2.9	41.3	2408
	516	13	12	3.3	44.1	2646
	517	13	13	3.8	47.1	2884
	518	13	14	4.2	49.9	3122
Fig. 161.	519	13	15	4.6	52.7	3360
	520	13	16	5.1	55.7	3598
	521	13	17	5.5	58.5	3850
	522	13	18	5.9	61.3	4088
	523	14	9	1.9	36.8	2142
	524	14	10	2.3	39.6	2408
	525	14	11	2.7	42.4	2674
	526	14	12	3.2	45.4	2940
	527	14	13	3.6	48.2	3206
	528	14	14	4.1	52.2	3472

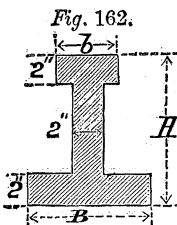
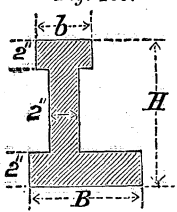
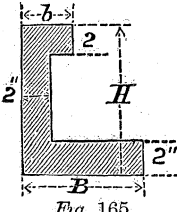
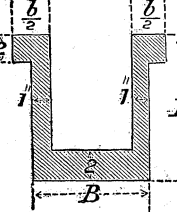
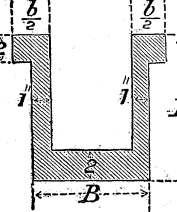


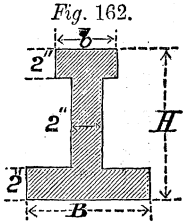
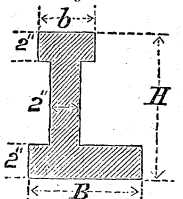
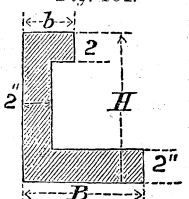
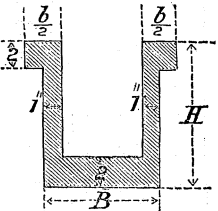
Number of section.	Height <i>H</i> in inches.	Width <i>B</i> of lower flange in inches.	Width <i>b</i> of upper flange in inches.	Sectional area in square inches.	Coefficient <i>K</i> 1.
529	14	15	4.5	54.0	3738
530	14	16	4.9	56.8	4004
531	14	17	5.4	59.8	4270
532	14	18	5.8	62.6	4536
533	15	9	1.7	37.9	2352
534	15	10	2.2	40.9	2646
535	15	11	2.6	43.7	2940
536	15	12	3.0	46.5	3234
537	15	13	3.5	49.5	3528
538	15	14	3.9	52.3	3822
539	15	15	4.4	55.3	4116
540	15	16	4.8	58.1	4410
541	15	17	5.3	61.1	4704
542	15	18	5.7	63.9	4998
543	16	9	1.6	39.2	2562
544	16	10	2.0	42.0	2884
545	16	11	2.5	45.0	3206
546	16	12	2.9	47.8	3528
547	16	13	3.4	50.8	3850
548	16	14	3.8	53.6	4172
549	16	15	4.3	56.6	4494
550	16	16	4.7	59.4	4816
551	16	17	5.2	65.4	5138
551	16	18	5.6	62.2	5460
552	17	10	1.9	43.3	3150

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
<p><i>Fig. 158.</i></p>	554	17	11	2.3	46.1	3486
	555	17	12	2.8	49.1	3836
	556	17	13	3.2	51.9	4186
	557	17	14	3.7	54.9	4536
	558	17	15	4.1	57.7	4872
<p><i>Fig. 159.</i></p>	559	17	16	4.6	60.7	5222
	560	17	17	5.0	63.5	5572
	561	17	18	5.5	66.5	5922
	562	18	10	1.6	44.2	3346
	563	18	11	2.1	47.2	3724
<p><i>Fig. 160.</i></p>	564	18	12	2.6	50.2	4102
	565	18	13	3.0	53.0	4480
	566	18	14	3.5	56.0	4868
	567	18	15	3.9	58.8	5236
	568	18	16	4.4	61.8	5628
<p><i>Fig. 161.</i></p>	569	18	17	4.9	64.8	6006
	570	18	18	5.3	67.6	6384

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
 <p>Fig. 162.</p>	571	6	9	2.3	26.6	504
	572	6	10	2.7	29.4	574
	573	6	11	3.0	32.0	630
	574	6	12	3.3	34.6	700
	575	6	13	3.7	37.4	756
	576	6	14	4.0	40.0	826
 <p>Fig. 163.</p>	577	6	15	4.3	42.6	882
	578	6	16	4.7	45.5	952
	579	6	17	5.0	48.0	1008
	580	6	18	5.3	50.6	1064
	581	7	9	2.3	28.6	686
	582	7	10	2.7	31.4	770
 <p>Fig. 164.</p>	583	7	11	3.0	34.0	854
	584	7	12	3.4	36.8	938
	585	7	13	3.8	39.6	1036
	586	7	14	4.1	42.2	1120
	587	7	15	4.5	45.0	1204
	588	7	16	4.9	47.8	1288
 <p>Fig. 165.</p>	589	7	17	5.2	50.4	1372
	590	7	18	5.6	53.2	1456
	591	8	9	2.2	30.4	868
	592	8	10	2.6	33.2	980
	593	8	11	2.9	35.8	1092
	594	8	12	3.3	38.6	1204
	595	8	13	3.7	41.4	1302
	596	8	14	4.1	44.2	1414
	597	8	15	4.5	47.0	1526

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
<p>Fig. 162.</p>	598	8	16	4.9	49.8	1638
	599	8	17	5.3	52.6	1750
	600	8	18	5.7	55.4	1848
	601	9	9	2.1	32.2	1064
	602	9	10	2.5	35.0	1204
	603	9	11	2.9	37.8	1330
<p>Fig. 163.</p>	604	9	12	3.3	40.6	1470
	605	9	13	3.7	43.4	1596
	606	9	14	4.1	46.2	1736
	607	9	15	4.5	49.0	1876
	608	9	16	4.9	51.8	2002
	609	9	17	5.3	54.6	2142
<p>Fig. 164.</p>	610	9	18	5.7	57.4	2282
	611	10	10	2.4	36.8	1414
	612	10	11	2.8	39.6	1582
	613	10	12	3.2	42.4	1736
	614	10	13	3.6	45.2	1904
	615	10	14	4.0	48.0	2058
<p>Fig. 165.</p>	616	10	15	4.4	50.8	2226
	617	10	16	4.8	53.6	2380
	618	10	17	5.2	56.4	2595
	619	10	18	5.7	59.4	2702
	620	11	10	2.2	38.4	1638
	621	11	11	2.6	41.2	1820
	622	11	12	3.0	44.0	2016
	623	11	13	3.5	47.0	2198
	624	11	14	3.9	49.8	2380

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K_1 .
 <p>Fig. 162.</p>	625	11	15	4.3	52.6	2576
	626	11	16	4.7	55.4	2758
	627	11	17	5.1	58.2	2954
	628	11	18	5.6	61.2	3136
	629	12	11	2.4	42.8	2086
 <p>Fig. 163.</p>	630	12	12	2.9	45.8	2296
	631	12	13	3.3	48.1	2506
	632	12	14	3.7	51.4	2716
	633	12	15	4.1	54.2	2940
	634	12	16	4.6	57.2	3150
 <p>Fig. 164.</p>	635	12	17	5.0	60.0	3360
	636	12	18	5.4	62.8	3570
	637	13	11	2.2	44.4	2338
	638	13	12	2.7	47.4	2576
	639	13	13	3.1	50.2	2814
 <p>Fig. 165.</p>	640	13	14	3.5	53.0	3052
	641	13	15	4.0	56.0	3290
	642	13	16	4.4	58.8	3528
	643	13	17	4.9	61.8	3780
	644	13	18	5.3	64.6	4018
	645	14	11	2.0	46.0	2604
	646	14	12	2.5	49.0	2870
	647	14	13	2.9	51.8	3136
	648	14	14	3.4	54.8	3402
	649	14	15	3.8	57.6	3668
	650	14	16	4.2	60.4	3934
	651	14	17	4.7	63.4	4208

	Number of section.	Height H in inches.	Width B of lower flange in inches.	Width b of upper flange in inches.	Sectional area in square inches.	Coefficient K .
 <p>Fig. 162.</p>	652	14	18	5.1	66.2	4452
	653	15	12	2.3	50.6	3164
	654	15	13	2.7	54.4	3444
	655	15	14	3.2	56.4	3738
	656	15	15	3.6	59.2	4032
	657	15	16	4.1	62.2	4296
	658	15	17	4.5	65.0	4606
 <p>Fig. 163.</p>	659	15	18	4.9	67.8	4900
	660	16	13	2.5	55.0	3742
	661	16	14	3.0	58.0	4074
	662	16	15	3.4	60.8	4396
	663	16	16	3.9	63.8	4718
	664	16	17	4.3	66.6	5026
	665	16	18	4.8	69.6	5348
 <p>Fig. 164.</p>	666	17	13	2.3	56.6	4060
	667	17	14	2.8	59.6	4410
	668	17	15	3.2	62.4	4760
	669	17	16	3.7	65.4	5110
	670	17	17	4.1	68.2	5460
	671	17	18	4.6	71.2	5810
	672	18	13	2.1	58.2	4382
 <p>Fig. 165.</p>	673	18	14	2.5	61.0	4746
	674	18	15	3.0	64.0	5124
	675	18	16	3.4	66.8	5502
	676	18	17	3.9	69.8	5080
	677	18	18	4.4	72.8	6258

STRENGTH OF WOODEN BEAMS.

Capacity W in lbs. of American white and yellow pine beams, joists, &c., from 1" x 1" to 15 x 15 in.

The modulus of rupture is taken at $\frac{10000}{8} = 1250$ lbs., or 8 times safety.

K' = tabulated coefficient, to be divided by
 l = distance between supports in inches, or length of beams in inches from support to free end of beam.

Thickness in inches.	Coefficient						
	Height in						
	1	2	3	4	5	6	7
1	1666	6666	15000	26666	41666	60000	81666
1½	2500	10000	22500	39999	62499	90000	122499
2	3333	13333	30000	53333	83333	120000	163333
2½	4166	16666	37500	66666	104166	150000	204166
3	5000	19999	45000	80000	124999	180000	244999
3½	5833	23333	52700	93333	145833	210000	285833
4	6666	26666	60000	106666	166666	240000	326666
4½	7499	29999	67500	119999	187499	270000	367499
5	8333	33333	75000	133333	208333	300000	408333
5½	9166	36666	82500	146666	229166	330000	449166
6	10000	39999	90000	159999	249999	360000	489999
6½	10833	43333	97500	173333	270833	390000	530833
7	11666	46666	105000	186666	291666	420000	571666
7½	12500	49999	112600	199999	312499	450000	612499
8	13333	53333	120000	213333	333333	480000	653333
8½	14166	56666	127500	226666	354166	510000	694166
9	14998	59999	135000	239999	374999	540000	734999
9½	15831	63333	142500	253333	395833	570000	775833
10	16666	66666	150000	266666	416666	600000	816666
10½	17500	69999	157500	279999	437499	630000	857599
11	18333	73333	165000	293333	458333	660000	898533
11½	19166	76666	172500	306666	479166	690000	939366
12	20000	79999	180000	319999	499999	720000	979999
12½	20833	83333	187500	333333	520833	750000	1020833
13	21666	86666	195000	346666	541666	780000	1061666
13½	22500	89999	202500	359999	562499	810000	1102499
14	23333	93333	210000	373333	583333	840000	1143333
14½	24166	96666	217500	386666	604166	870900	1184166
15	25000	99999	225000	399999	624999	900000	1224999

BEAMS SUPPORTED AT THE ENDS.

Load equally distributed, $W = \frac{K'}{l}$ or $K' = lW$. 1

Load concentrated at centre, $W = \frac{K'}{2l}$ or $K' = 2lW$. 2

BEAMS FIXED AT ONE END.

Load equally distributed, $W = \frac{K'}{4l}$ or $K' = 4lW$. 3

Load concentrated at free end, $W = \frac{K'}{8l}$ or $K' = 8lW$. 4

K'.

inches.

8	9	10	11	12	13	14	15
106666	135000	166666	201757	240000	281666	326666	375000
159999	202500	249999	302636	360000	422499	489999	562500
213333	270000	333333	403515	480000	563333	653333	750000
266666	337500	416666	504393	600000	704166	816666	937500
319999	405000	499999	605272	720000	844999	979999	1125000
373333	472500	583333	706151	840000	985833	1143333	1312500
426666	540000	666666	807030	960000	1126666	1306666	1500000
479999	607500	749999	907908	1080000	1267499	1469999	1687500
533333	675000	833333	1008787	1200000	1408333	1633333	1875000
586666	742500	916666	1109666	1320000	1549166	1796666	2062500
639999	810000	999999	1210545	1440000	1689999	1959999	2250000
693333	877500	1083333	1311423	1560000	1830833	2123333	2437500
746666	945000	1166666	1412302	1680000	1971666	2286666	2625000
799999	1012500	1249999	1513181	1800000	2112499	2449999	2812500
853333	1080000	1333333	1614060	1920000	2253333	2613333	3000000
906666	1147500	1416666	1714938	2040000	2394166	2776666	3187500
959999	1215000	1499999	1815817	2160000	2534999	2939999	3375000
1013333	1282500	1583333	1916696	2280000	2675833	3103333	3562500
1066666	1350000	1666666	2017575	2400000	2816666	3266666	3750000
1119999	1417500	1749999	2118453	2520000	2957499	3429999	3937500
1173333	1485000	1833333	2219332	2640000	3098333	3593333	4125000
1226666	1552500	1916666	2320211	2760000	3239166	3756666	4312500
1279999	1620000	1999999	2421090	2880000	3379999	3919999	4500000
1333333	1687500	2083333	2521968	3000000	3520833	4083333	4687500
1386666	1755000	2166666	2622847	3120000	3661666	4246666	4875000
1439999	1822500	2249999	2723726	3240000	3802499	4409999	5062500
1493333	1890000	2333333	2824605	3360000	3943333	4573333	5250000
1546666	1957500	2416666	2925483	3480000	4084166	4736666	5437500
1599999	2025000	2499999	3026362	3600000	4224999	4899999	5625000

PRESSURE ON SUPPORTS.

REACTION OF SUPPORTS.

For a continuous beam, horizontal or inclined. Load W , equally distributed, and supports equal distance apart. Applicable to trussed beams, rafters, or beams supported by three or more supports.

Reference. (Fig. 166.)

W = Weight of load per unit of length in lbs.

L = Distance between supports in units of length.

P, P_1, P_2 = Pressure on supports in lbs., counting from end support to center of beam.

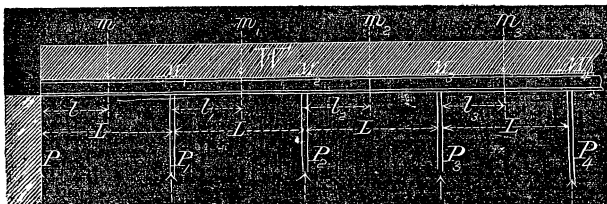
M, M_1, M_2 = Moments of rupture over supports.

m, m_1, m_2 = Moments of rupture between supports.

l, l_1, l_2 = The distance from a support to section where moments m, m_1, m_2 occur.

By this table the pressure upon any support, from 3 to 9 in number, can be ascertained; also the moments of rupture. The table is used in calculating the strains in roof trusses, &c.

Fig. 166.



Reactions or pressure.	Number of Supports.				
	3	4	5	7	9
P	$0.375 \frac{W}{L}$	$0.4 \frac{W}{L}$	$0.3929 \frac{W}{L}$	$0.3942 \frac{W}{L}$	$0.3943 \frac{W}{L}$
P_1	$1.25 \frac{W}{L}$	$1.1 \frac{W}{L}$	$1.1429 \frac{W}{L}$	$1.1346 \frac{W}{L}$	$1.1340 \frac{W}{L}$
P_2	$0.9286 \frac{W}{L}$	$0.9615 \frac{W}{L}$	$0.9629 \frac{W}{L}$
P_3	$1.0192 \frac{W}{L}$	$1.0103 \frac{W}{L}$
P_4	$0.9948 \frac{W}{L}$
M_1	$0.125 \frac{W}{L^2}$	$0.1 \frac{W}{L^2}$	$0.1071 \frac{W}{L^2}$	$0.1058 \frac{W}{L^2}$	$0.1057 \frac{W}{L^2}$
M_2	$0.0714 \frac{W}{L^2}$	$0.0769 \frac{W}{L^2}$	$0.0773 \frac{W}{L^2}$
M_3	$0.0865 \frac{W}{L^2}$	$0.0850 \frac{W}{L^2}$
M_4	$0.0824 \frac{W}{L^2}$

Reactions or pressure.	Number of Supports.				
	3	4	5	7	9
m	$0.0703 W, L^2$	$0.08 W, L^2$	$0.0772 W, L^2$	$0.0777 W, L^2$	$0.0777 W, L^2$
m_1	$0.025 W, L^2$	$0.0364 W, L^2$	$0.0340 W, L^2$	$0.0339 W, L^2$
m_2	$0.0434 W, L^2$	$0.0438 W, L^2$
m_3	$0.0412 W, L^2$
l	$0.375 L$	$0.4 L$	$0.3928 L$	$0.3942 L$	$0.3943 L$
l_1	$0.5 L$	$0.535 L$	$0.5288 L$	$0.5283 L$
l_2	$0.4903 L$	$0.4922 L$
l_3	$0.5025 L$

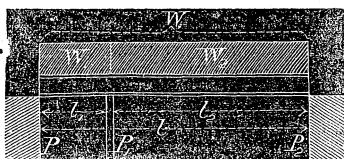
Reference. (Figs. 167, 168, and 169.)

W, W_1, W_2 = Load in lbs.

l, l_1, l_2 = Dimensions in units of length.

P, P_1, P_2 = Pressure on supports in lbs.

Fig. 167.



Three supports,
unequal distances
apart.

Load equally distributed:

$$l_1 < l_2;$$

$$W_1 = \frac{l_1}{l} W$$

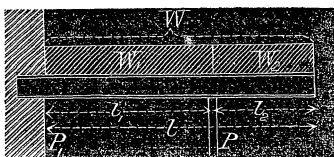
$$W_2 = \frac{l_2}{l} W$$

$$P = \frac{3}{8} W_1 = \frac{3}{8} \frac{l_1}{l} W$$

$$P_1 = \frac{5}{8} (W_1 + W_2) = \frac{5}{8} W$$

$$P_2 = \frac{3}{8} W_2 = \frac{3}{8} \frac{l_2}{l} W$$

Fig. 168.



One support, and
fixed at one end.

Load equally distributed:

$$l_1 > l_2$$

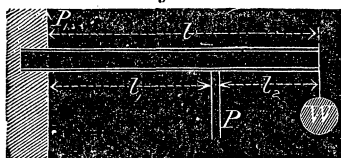
$$W_1 = \frac{l_1}{l} W$$

$$W_2 = \frac{l_2}{l} W$$

$$P = \frac{1}{2} \frac{Wl}{l_1}$$

$$P_1 = W - P = \left(1 - \frac{1}{2} \frac{l}{l_2}\right) W$$

Fig. 169.



One support,
and fixed at
one end.

Load concentrated at free end:

$$P = \frac{l}{l_1} W$$

$$P_1 = P - W = \left(\frac{l}{l_1} - 1 \right) W = \frac{l_2}{l_1} W$$

COMPRESSIVE STRAIN AND PRESSURE ON SUPPORTS.

SLOPING BEAMS, RAFTERS, &c.

Load W equally distributed.

For the cross-breaking strain, the rafter, &c., is to be treated as a horizontal beam of the length l . (See *Compound Strains in Beam, &c.*)

Reference.

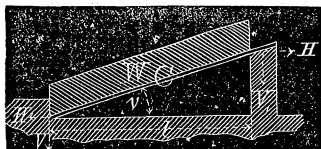
C = Compression in direction of beam.

H = Horizontal strain acting on support.

V = Pressure on supports.

Lower end supported vertically and horizontally; upper end resting on inclined support:

Fig. 170.



$$C = \frac{W}{2} \sin.v$$

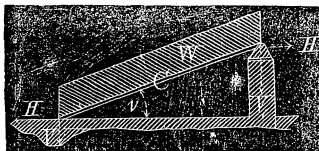
$$V = W - V_1 = W \left(1 - \frac{1}{2} (\cos.v)^2 \right)$$

$$H = \frac{W}{2} \sin.v \cos.v$$

$$V_1 = \frac{W}{2} (\cos.v)^2$$

Upper end fixed; lower end supported horizontally :

Fig. 171.



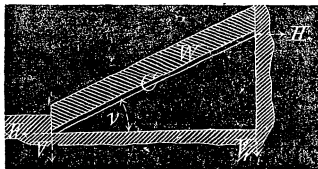
$$C = 0$$

$$H = 0$$

$$V = V_1 = \frac{W}{2}$$

Upper end resting against a vertical surface; lower end supported vertically and horizontally :

Fig. 172.



$$C = \frac{W}{2 \sin. v}$$

$$H = \frac{W}{2} \cotg v$$

$$V = W$$

$$V_1 = 0$$

RESISTANCE TO CRUSHING.

STRENGTH OF COLUMNS, PILLARS, AND STRUTS.

Reference.

A = Area of cross-section in inches.

C = Coefficient, depending on the material.

I = Least moment of inertia of cross-section.

W = Capacity of column, pillar, or strut in lbs.

a = Coefficient, depending on the material in respect to flexure.

c = Coefficient, depending on the material.

h = The least dimension across the section in inches.

k = Factor of safety.

l = Length of column, &c., in inches.

r = Least radius of gyration.

To find the square of the radius of gyration (r^2) of a plane about a given axis, divide the least moment of inertia by the sectional area of the plane; that is, $r^2 = \frac{I}{A}$.

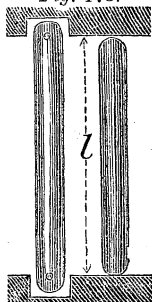
Values of—	For Malleable Iron.	For Cast Iron.	For Dry Timber.
$C =$	36,000 lbs.	80,000 lbs.	7,200 lbs.
$c =$	36,000 "	3,200 "	3,000 "
$a =$	0.000333	0.0025	0.004

The factor of safety k should be, for wrought iron = 6; for cast iron = 8; for timber = 10. This applies to moving loads.

Case 1.

Rounded or hinged at both ends, as per—

Fig. 173.



For square, rectangular, or circular cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + 4a \frac{l^2}{h^2}}$$

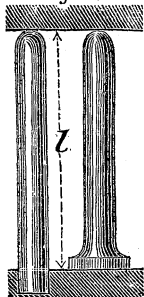
For any other cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + \frac{4l^2}{cr^2}}$$

Case 2.

Fixed, or having a flat base at one end, and rounded or hinged at the other, as per—

Fig. 174.



For square, rectangular, or circular cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + 2a \frac{l^2}{h^2}}$$

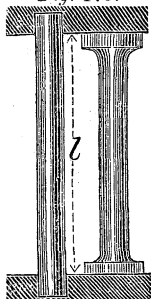
For any other cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + \frac{16.l^2}{9.c.r^2}}$$

Case 3.

Fixed, or having flat bases at both ends, as per—

Fig. 175.



For square, rectangular, or circular cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + a \frac{l^2}{h^2}}$$

For any other cross-section:

$$W = \frac{1}{k} \frac{CA}{1 + \frac{l^2}{c \cdot r^2}}$$

EXAMPLES.

Case 1.

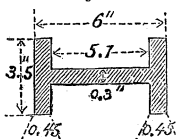
Rounded at both ends:

What is the capacity of a wrought-iron strut of the annexed figure and dimensions?

$l = 10 \text{ feet} = 120 \text{ inches.}$

$A = 4.68 \text{ inches.}$

Fig. 176.



$$I = \frac{0.9 \times 3.5^3 + 5.1 \times 0.3^3}{12} = 3.227$$

$$r^2 = \frac{3.227}{4.68} = 0.689$$

$$W = \frac{1}{4} \frac{36000 \times 4.68}{1 + \frac{4 \times 120^2}{36000 \times 0.689}} = \frac{1}{4} \frac{168480}{1 + \frac{57600}{24804}} =$$

$$\frac{1}{4} \frac{168480}{3.322} = 12,679 \text{ lbs.}$$

The same as above, in Case 3, fixed at both ends:

$$W = \frac{1}{4} \frac{36000 \times 4.69}{1 + \frac{120^2}{36000 \times 0.689}} = \frac{1}{4} \frac{168480}{1 + \frac{14400}{24804}} =$$

$$\frac{1}{4} \frac{168480}{1.58} = 26,677 \text{ lbs.}$$

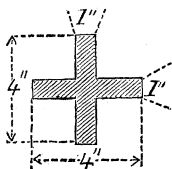
For the annexed figure and dimensions; otherwise, same as above:

$$A = 7 \text{ inches.}$$

Case 1.

Rounded at both ends:

Fig. 177.



$$I = \frac{1 \times 4^3 + 3 \times 1^3}{12} = 5.6$$

$$r^2 = \frac{5.6}{7} = 0.8$$

$$W = \frac{1}{4} \frac{36000 \times 7}{1 + \frac{4 \times 120^2}{36000 \times 0.8}} = \frac{1}{4} \frac{252000}{3} = 21,000 \text{ lbs.}$$

Same as above, in Case 3, fixed at both ends:

$$W = \frac{1}{4} \frac{36000 \times 7}{1 + \frac{120^2}{36000 \times 0.8}} = \frac{1}{4} \frac{252000}{1.5} = 42,000 \text{ lbs.}$$

Case 3.

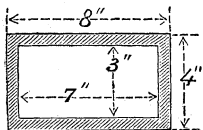
Fixed ends:

What is the capacity of a *cast-iron* pillar of the annexed figure and dimensions?

$$l = 10 \text{ feet} = 120 \text{ inches.}$$

$$A = 11 \text{ inches.}$$

Fig. 178.

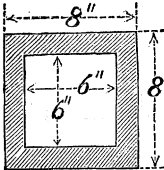


$$I = \frac{8 \times 4^3 - 7 \times 3^3}{12} = 26.9$$

$$W = \frac{1}{8} \frac{80000 \times 11}{1 + 0.0025 \frac{120^2}{4^2}} = \frac{1}{8} \frac{880000}{3.25} = 33,846 \text{ lbs.}$$

For the annexed figure and dimensions; otherwise, same as above.

Fig. 179.



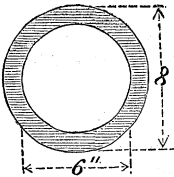
$$A = 28 \text{ inches.}$$

$$W = \frac{1}{8} \frac{80000 \times 28}{1 + 0.0025 \frac{120^2}{8^2}} =$$

$$\frac{1}{8} \frac{2240000}{1.5625} = 179,200 \text{ lbs.}$$

For the annexed figure and dimensions; otherwise, same as above.

Fig. 180.



$$A = 22 \text{ inches.}$$

$$W = \frac{1}{8} \frac{80000 \times 22}{1 + 0.0025 \frac{120^2}{8^2}} =$$

$$\frac{1}{8} \frac{1760000}{1.5625} = 140,800 \text{ lbs.}$$

To find the capacity of a Column, Pillar, or Strut of any cross-section by the following Table :

Find how many times the least dimension h across the section is contained in the length l of column, &c.—that is, $\frac{l}{h}$ —then

multiply the corresponding number on the same horizontal line, under K'' , by the sectional area of cross-section. This gives the capacity in tons of 2,000 lbs.

Let l = Length of column, &c.

h = Least dimension of cross-section.

K'' = Capacity in tons of one square inch of cross-section, to be multiplied by sectional area of desired cross-section.

Various sections for which this table is applicable:

Fig. 181.

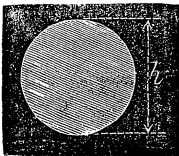


Fig. 182.

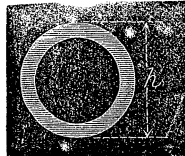


Fig. 183.

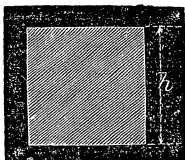


Fig. 184.

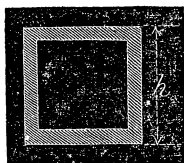


Fig. 185.

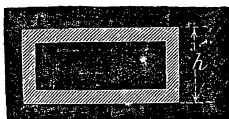


Fig. 186.

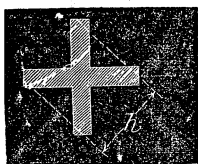


Fig. 187.

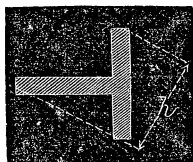
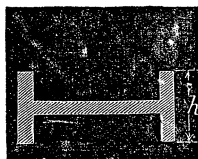


Fig. 188.

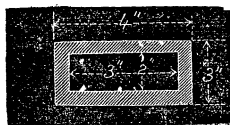


[NOTE.—This table is strictly correct, only for columns, &c., with circular or rectangular cross-section. As the error is small, it may be used for any cross-section.]

Example explanatory of the following table.

What is the capacity of a cast-iron column 10 feet = 120 inches long, fixed at both ends, and of the annexed cross-section and dimensions?

Fig. 189.



$$\frac{l}{h} = \frac{120}{3} = 40 \quad K'' \text{ for } 40 = 1,000 \text{ tons.}$$

Area = 6 inches.

$$W = 6 \times 1 = 6 \text{ tons, 8 times safety.}$$

Column, &c., fixed at both ends.

Cast Iron—eight times safety.						Wrought Iron—six times safety.					
$\frac{l}{h}$	K''	$\frac{l}{h}$	K''	$\frac{l}{h}$	K''	$\frac{l}{h}$	K''	$\frac{l}{h}$	K''	$\frac{l}{h}$	K''
	Tons.		Tons.		Tons.		Tons.		Tons.		Tons.
1	4.987	25	1.951	49	0.714	1	2.999	25	2.487	49	1.674
2	4.950	26	1.858	50	0.689	2	2.996	26	2.452	50	1.644
3	4.890	27	1.771	51	0.666	3	2.991	27	2.418	51	1.615
4	4.807	28	1.689	52	0.644	4	2.984	28	2.383	52	1.585
5	4.705	29	1.611	53	0.623	5	2.975	29	2.348	53	1.557
6	4.587	30	1.538	54	0.603	6	2.964	30	2.313	54	1.529
7	4.450	31	1.469	55	0.584	7	2.953	31	2.277	55	1.501
8	4.310	32	1.404	56	0.565	8	2.938	32	2.242	56	1.474
9	4.158	33	1.343	57	0.548	9	2.921	33	2.206	57	1.448
10	4.000	34	1.285	58	0.531	10	2.905	34	2.172	58	1.422
11	3.838	35	1.230	59	0.515	11	2.885	35	2.136	59	1.396
12	3.676	36	1.179	60	0.500	12	2.863	36	2.101	60	1.371
13	3.514	37	1.130	61	0.485	13	2.841	37	2.067	61	1.347
14	3.355	38	1.084	62	0.471	14	2.817	38	2.032	62	1.323
15	3.200	39	1.041	63	0.457	15	2.792	39	1.998	63	1.299
16	3.048	40	1.000	64	0.445	16	2.766	40	1.963	64	1.276
17	2.902	41	0.961	65	0.432	17	2.738	41	1.930	65	1.253
18	2.762	42	0.924	66	0.420	18	2.711	42	1.896	66	1.228
19	2.628	43	0.889	67	0.409	19	2.680	43	1.863	67	1.209
20	2.500	44	0.856	68	0.398	20	2.650	44	1.831	68	1.187
21	2.378	45	0.824	69	0.387	21	2.619	45	1.798	69	1.167
22	2.262	46	0.794	70	0.377	22	2.586	46	1.767	70	1.146
23	2.152	47	0.766	71	0.367	23	2.554	47	1.735	71	1.126
24	2.049	48	0.739	72	0.358	24	2.520	48	1.704	72	1.107

Strength of Columns, Pillars, or Struts, of seasoned wood, round or square section.

Fixed at both ends. All dimensions in inches.

Find how many times the least dimension across the section is contained in the length or height of column, &c.; that is, $\frac{H}{D}$;

then multiply the corresponding figures on the same horizontal line under K'' by the sectional area of cross-section. This gives the capacity of column, &c., in tons of 2,000 lbs., 10 times safety.

Reference.

H = Length of column, &c.

D = Least dimension of cross-section.

K'' = Capacity in tons of one square inch of cross-section, to be multiplied by sectional area of desired cross-section.

The coefficient C for white and yellow pine in the following table is taken at $\frac{6000}{10} = 600$ lbs. for safety:

For oak at $\frac{8000}{10} = 800$ lbs. per square inch for safety.

EXAMPLE.—What is the capacity of a pillar of oak, section 4×6 inches, length = 12 feet = 144 inches?

$$\frac{H}{D} = \frac{144}{4} = 36, K'' \text{ for } 36 = 0.064 \times 4 \times 6 = 1.536 \text{ tons.}$$

Capacity K'' of one square inch in tons of 2,000 lbs.

White and Yellow Pine.				Oak.			
$\frac{H}{D} =$	K''	$\frac{H}{D} =$	K''	$\frac{H}{D} =$	K''	$\frac{H}{D} =$	K''
1	0.299	26	0.081	1	0.399	26	0.108
2	0.295	27	0.076	2	0.394	27	0.102
3	0.289	28	0.072	3	0.386	28	0.096
4	0.282	29	0.068	4	0.376	29	0.091
5	0.272	30	0.065	5	0.363	30	0.086
6	0.262	31	0.061	6	0.349	31	0.082
7	0.251	32	0.058	7	0.334	32	0.078
8	0.239	33	0.056	8	0.319	33	0.074
9	0.226	34	0.053	9	0.302	34	0.071
10	0.214	35	0.050	10	0.285	35	0.067
11	0.202	36	0.048	11	0.269	36	0.064
12	0.190	37	0.046	12	0.254	37	0.061
13	0.179	38	0.044	13	0.238	38	0.059
14	0.168	39	0.042	14	0.224	39	0.056
15	0.158	40	0.040	15	0.210	40	0.054
16	0.148	41	0.038	16	0.197	41	0.051
17	0.139	42	0.037	17	0.185	42	0.049
18	0.130	43	0.035	18	0.174	43	0.047
19	0.123	44	0.034	19	0.163	44	0.045
20	0.115	45	0.033	20	0.154	45	0.044
21	0.108	46	0.031	21	0.144	46	0.042
22	0.102	47	0.030	22	0.136	47	0.040
23	0.096	48	0.029	23	0.128	48	0.039
24	0.090	49	0.028	24	0.121	49	0.037
25	0.085	50	0.027	25	0.114	50	0.036

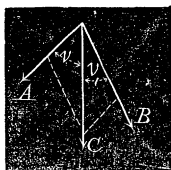
PARALLELOGRAM OF FORCES.

COMPOSITION AND RESOLUTION OF FORCES.

Reference.

A, B, C = Forces, or strains, acting on a single point.
 v, v' , = angles.

Fig. 190.



$$A = \frac{C \sin. v'}{\sin. (v + v')},$$

$$B = \frac{C \sin. v}{\sin. (v + v')}, \text{ when } v = v', A = B = \frac{C}{2} \sec. v;$$

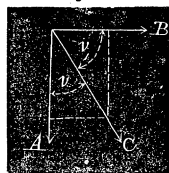
when $v + v' < 90^\circ$

$$C = \sqrt{A^2 + B^2 + (2 A B \cos. (v + v'))}$$

when $v + v' > 90^\circ$

$$C = \sqrt{A^2 + B^2 - [2 A B \cos. (180^\circ - (v + v'))]}$$

Fig. 191.



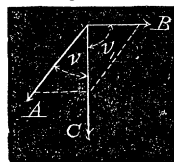
$v + v' = 90^\circ$

$$A = C \cos. v$$

$$B = C \sin. v = C \cos. v'$$

$$C = \sqrt{A^2 + B^2}$$

Fig. 192.



$v' = 90^\circ$

$$A = \frac{C}{\cos. v}$$

$$B = C \tan. v$$

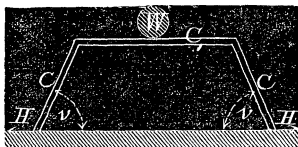
$$C = \sqrt{A^2 - B^2}$$

STRAINS IN FRAMES.

Reference. C = Compressive strain in units of weight. T = Tensile " " V = Vertical " " H = Horizontal " " W = Load in units of weight. l = Dimensions in units of length. v = Angle between horizontal and inclined member.

For cross-breaking strain, see "Resistance to cross-breaking."

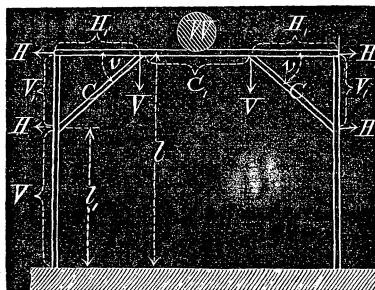
Fig. 193.



$$C = \frac{W}{2 \sin. v}$$

$$C_l = \frac{W}{2} \cotg. v = H$$

Fig. 194.



$$C = \frac{11}{16} \frac{W}{\sin. v}$$

$$C_l = H = \frac{11}{16} W \cotg. v = \text{cross-breaking strain at } H.$$

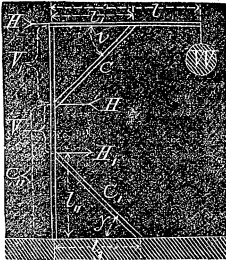
$$H_l = \frac{l_l}{l} H = \frac{11}{16} \cdot \frac{l_l}{l} W \cotg. v = \text{tension in } H_l.$$

$$H - H_l = \frac{11}{16} \cdot \left(\frac{l - l_l}{l} \right) W \cotg. v = \text{compression in } C_l$$

$$V = \frac{11}{16} W.$$

$$V_l = \frac{11}{16} W.$$

Fig. 195.



$$C = \frac{lW}{l' \sin. v} = \text{compression.}$$

$$C' = \frac{H'}{\cos. y} = \frac{W.l}{l'' \cos. y} = \text{compression.}$$

$$C'' = \frac{W}{H} = W.l$$

$$H' = \frac{W.l}{l''}$$

$$V = H' \tan g. y = \frac{W.l}{l''} \tan g. y$$

When $l > l_3$ the portion l'' is in tension $= V - W =$

$$W \left(\frac{l}{l''} \tan g. y - 1 \right)$$

When $l < l_3$ the portion l'' is in compression $= W - V =$

$$W \left(1 - \frac{l}{l''} \tan g. y \right)$$

$$V' = \frac{l - l'}{l'} \cdot W = \text{tension.}$$

Fig. 196.

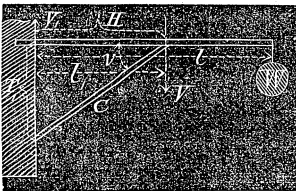
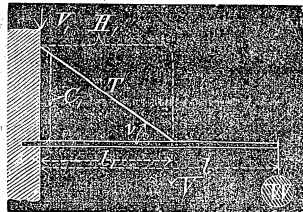


Fig. 197.



Ends of beams built into wall or fixed:

$$V = \frac{l}{l'} W$$

$$V' = V - W = \left(\frac{l - l'}{l'} \right) W, \quad T' (\text{tension}) = C' (\text{compression.})$$

$$C = \left(\frac{3l - l'}{2l'} \right) \frac{W}{\sin. v} = (\text{compression}) = T (\text{tension.})$$

$$H = \left(\frac{3l - l'}{2l'} \right) W \cotg. v = (\text{tension}) = H' (\text{compression.})$$

Ends of beams *not* built into wall or fixed:

$$V = \frac{l}{l'} W$$

$$V' = V - W = \left(\frac{l - l'}{l'} \right) W = C' (\text{compression}) = T' (\text{tension.})$$

$$C = \frac{V}{\sin. v} = \frac{l W}{l' \sin. v} = T (\text{tension.})$$

$$H = V \cotg. v = \frac{l}{l'} W \cotg. v = (\text{tension}) = H' (\text{compression.})$$

STRAINS IN BOOM DERRICKS.

Reference.

C = Compression in boom.

C' = Compression in mast.

T = Tension in tackling.

T' = Tension in guy.

t = Tension in runner from mast head to weight.

t' = Tension in runner from boom head to weight.

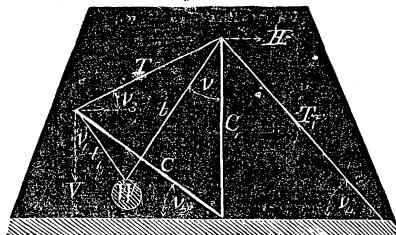
W = Weight or load.

H = Horizontal strain.

V = Vertical strain.

v, v_1, v_2 = Angles. (See Figure.)

Fig. 198.



$$t = \frac{W \sin. v_1}{\sin. (v + v_1)}$$

$$V = t_1 \cosin. v_1$$

$$C = V \operatorname{cosec}. v_2$$

$$T = V \operatorname{cosec}. v_3$$

$$t_1 = \frac{W \sin. v}{\sin. (v + v_1)}$$

$$H = V \cotg. v_3$$

$$C_1 = W$$

$$T_1 = V \cotg. v_3 \sec. v_4$$

STRAINS IN TRUSSES.

Load equally distributed.

Reference.

W = Load equally distributed in lbs.

l = Distance between abutments.

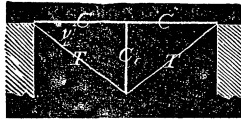
v = Angle between horizontal and diagonal.

C = Compression in lbs., (denoted by thick lines.)

T = Tension in lbs., (denoted by thin lines.)

$$2 \text{ Bays} = \frac{l}{2}$$

Fig. 199.



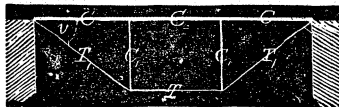
$$C = \frac{5}{16} W \cotg. v$$

$$C_1 = \frac{5}{8} W$$

$$T = \frac{5}{16} \frac{W}{\sin. v}$$

$$3 \text{ Bays} = \frac{l}{3}$$

Fig. 200.



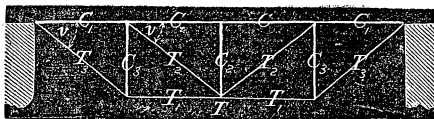
$$C = T = \frac{W}{3} \cotg. v$$

$$C_1 = \frac{W}{3}$$

$$T_1 = \frac{1}{3} \frac{W}{\sin. v}$$

$$4 \text{ Bays} = \frac{l}{4}$$

Fig. 201.



$$C = T = \frac{4 C_2}{5} \cotg. v$$

$$C_1 = T_1$$

$$C_2 = \frac{W}{4}$$

$$C_3 = \frac{3 C_2}{2}$$

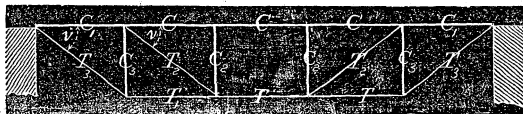
$$T_1 = \frac{3 C_2}{2} \cotg. v$$

$$T_2 = \frac{C_2}{2} \operatorname{cosec}. v$$

$$T_3 = 3 T_2$$

$$5 \text{ Bays} = \frac{l}{5}$$

Fig. 202.



$$C = T = 3 C_2 \cotg. v$$

$$C_1 = T_1 = 2 C_2 \cotg. v$$

$$C_2 = \frac{W}{5}$$

$$C_3 = 2 C_2$$

$$T_2 = C_2 \operatorname{cosec}. v$$

$$T_3 = 2 T_2$$

$$6 \text{ Bays} = \frac{l}{6}$$

Fig. 203.



$$C = T = \frac{9C_3}{2} \cotg. v$$

$$C_4 = \frac{3C_3}{2}$$

$$C_1 = T_1 = \frac{8C_3}{2} \cotg. v$$

$$C_5 = \frac{5C_3}{2}$$

$$C_2 = T_2 = \frac{5C_3}{2} \cotg. v$$

$$T_3 = \frac{C_3}{2} \operatorname{cosec}. v$$

$$C_3 = \frac{W}{6}$$

$$T_4 = 3T_3$$

$$T_5 = 5T_3$$

TABLE OF CONSTANTS, BASED ON FOREGOING FORMULA.

Load equally distributed.

Table of constants for strains in respective member of trusses, from 2 to 6 bays, with diagonals inclined from 5° to 45° :

Reference.

W = Load in lbs., equally distributed over whole length of truss, to be multiplied by constant for strain in respective member.

v = Angle between horizontal and diagonal.

C = Compression in lbs. in respective member.

T = Tension in lbs. in respective member.

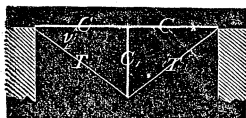
EXAMPLE.—Required, the strain in the various members of a truss of 4 bays. Length = 40 feet; load W = 80,000 lbs.; angle v = 20° .

Members.	Constants.	W .	Strains.
$C = T$	$= 1.372$	$\times 80,000$	$= 109,760$ lbs.
$C_1 = T_1$	$= 1.029$	$\times 80,000$	$= 82,320$ "
C_2	$= 0.25$	$\times 80,000$	$= 20,000$ "
C_3	$= 0.375$	$\times 80,000$	$= 30,000$ "
T_2	$= 0.365$	$\times 80,000$	$= 29,200$ "
T_3	$= 1.095$	$\times 80,000$	$= 87,600$ "

[NOTE.—When the trusses are inverted, the strains change in kind, but not in amount.]

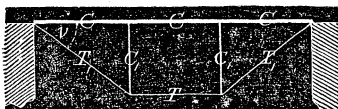
$$2 \text{ Bays} = \frac{l}{2}$$

Fig. 204.



$$3 \text{ Bays} = \frac{l}{3}$$

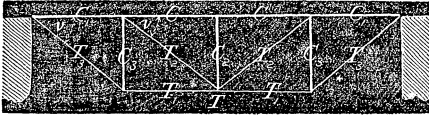
Fig. 205.



v	C	C_1	T	$C = T$	C_1	T_1
5	3.572	0.625	3.584	3.810	0.333	3.820
6	2.972	"	2.987	3.170	"	3.186
7	2.544	"	2.562	2.713	"	2.733
8	2.225	"	2.244	2.370	"	2.393
9	1.972	"	1.997	2.103	"	2.130
10	1.772	"	1.800	1.890	"	1.920
11	1.610	"	1.640	1.710	"	1.747
12	1.469	"	1.500	1.570	"	1.603
13	1.353	"	1.390	1.444	"	1.483
14	1.253	"	1.290	1.333	"	1.376
15	1.166	"	1.210	1.243	"	1.286
16	1.087	"	1.134	1.160	"	1.210
17	1.022	"	1.070	1.090	"	1.140
18	0.959	"	1.013	1.023	"	1.080
19	0.906	"	0.959	0.970	"	1.023
20	0.859	"	0.912	0.917	"	0.973
21	0.813	"	0.872	0.866	"	0.930
22	0.778	"	0.834	0.823	"	0.890
23	0.734	"	0.790	0.783	"	0.853
24	0.703	"	0.765	0.750	"	0.810
25	0.668	"	0.738	0.713	"	0.786
26	0.641	"	0.712	0.685	"	0.760
27	0.613	"	0.687	0.653	"	0.730
28	0.587	"	0.666	0.626	"	0.701
29	0.562	"	0.644	0.600	"	0.686
30	0.541	"	0.625	0.643	"	0.666
31	0.519	"	0.606	0.555	"	0.646
32	0.500	"	0.591	0.533	"	0.630
33	0.481	"	0.575	0.513	"	0.613
34	0.463	"	0.559	0.493	"	0.596
35	0.447	"	0.544	0.476	"	0.580
36	0.431	"	0.531	0.460	"	0.566
37	0.416	"	0.519	0.444	"	0.553
38	0.400	"	0.506	0.426	"	0.540
39	0.384	"	0.497	0.410	"	0.530
40	0.372	"	0.487	0.396	"	0.520
41	0.359	"	0.475	0.385	"	0.506
42	0.347	"	0.466	0.370	"	0.496
43	0.334	"	0.456	0.357	"	0.486
44	0.322	"	0.450	0.343	"	0.480
45	0.312	"	0.444	0.333	"	0.473

$$4 \text{ Bays} = \frac{l}{4}$$

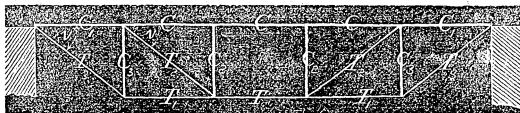
Fig. 206.



v	$C = T$	$C_1 = T_1$	C_2	C_3	T_2	T_3
5	5.720	4.290	0.250	0.375	1.434	4.032
6	4.730	3.570	"	"	1.200	3.600
7	4.068	3.051	"	"	1.025	3.075
8	3.560	2.670	"	"	0.897	2.591
9	3.164	2.373	"	"	0.799	2.397
10	2.832	2.124	"	"	0.720	2.160
11	2.568	1.926	"	"	0.655	1.965
12	2.388	1.791	"	"	0.601	1.803
13	2.164	1.623	"	"	0.556	1.668
14	2.000	1.500	"	"	0.516	1.548
15	1.864	1.398	"	"	0.482	1.446
16	1.740	1.305	"	"	0.454	1.362
17	1.632	1.224	"	"	0.428	1.284
18	1.532	1.149	"	"	0.405	1.215
19	1.448	1.086	"	"	0.384	1.152
20	1.372	1.029	"	"	0.365	1.095
21	1.300	0.975	"	"	0.349	1.047
22	1.236	0.927	"	"	0.334	1.002
23	1.172	0.879	"	"	0.320	0.960
24	1.124	0.843	"	"	0.306	0.918
25	1.068	0.801	"	"	0.295	0.885
26	1.024	0.768	"	"	0.285	0.855
27	0.980	0.735	"	"	0.275	0.825
28	0.940	0.705	"	"	0.266	0.798
29	0.900	0.675	"	"	0.258	0.774
30	0.864	0.648	"	"	0.250	0.750
31	0.828	0.621	"	"	0.243	0.729
32	0.800	0.600	"	"	0.236	0.708
33	0.768	0.576	"	"	0.230	0.690
34	0.740	0.555	"	"	0.224	0.672
35	0.720	0.540	"	"	0.218	0.654
36	0.688	0.516	"	"	0.212	0.636
37	0.664	0.498	"	"	0.207	0.621
38	0.640	0.480	"	"	0.203	0.609
39	0.616	0.462	"	"	0.199	0.597
40	0.600	0.450	"	"	0.195	0.585
41	0.576	0.432	"	"	0.190	0.570
42	0.560	0.420	"	"	0.186	0.558
43	0.536	0.402	"	"	0.183	0.549
44	0.520	0.390	"	"	0.180	0.540
45	0.500	0.375	"	"	0.177	0.531

$$5 \text{ Bays} = \frac{l}{5}$$

Fig. 207.



v	$C = T$	$C_1 = T_1$	C_2	C_3	T_2	T_3
5	6.858	4.572	0.200	0.400	2.294	4.588
6	5.706	3.804	"	"	1.912	3.824
7	4.884	3.256	"	"	1.640	3.280
8	4.272	2.848	"	"	1.436	2.872
9	3.786	2.524	"	"	1.278	2.556
10	3.402	2.268	"	"	1.152	2.304
11	3.084	2.056	"	"	1.048	2.096
12	2.820	1.880	"	"	0.962	1.924
13	2.598	1.732	"	"	0.890	1.780
14	2.406	1.604	"	"	0.826	1.652
15	2.238	1.492	"	"	0.772	1.544
16	2.088	1.392	"	"	0.726	1.452
17	1.962	1.308	"	"	0.684	1.368
18	1.842	1.228	"	"	0.648	1.296
19	1.740	1.160	"	"	0.614	1.228
20	1.650	1.100	"	"	0.584	1.168
21	1.560	1.040	"	"	0.558	1.116
22	1.482	0.988	"	"	0.534	1.068
23	1.410	0.940	"	"	0.512	1.024
24	1.350	0.900	"	"	0.490	0.980
25	1.284	0.856	"	"	0.472	0.944
26	1.230	0.820	"	"	0.456	0.912
27	1.176	0.784	"	"	0.440	0.880
28	1.128	0.752	"	"	0.426	0.852
29	1.080	0.720	"	"	0.412	0.824
30	1.038	0.692	"	"	0.400	0.800
31	0.996	0.664	"	"	0.388	0.776
32	0.960	0.640	"	"	0.378	0.756
33	0.924	0.616	"	"	0.368	0.736
34	0.888	0.592	"	"	0.358	0.716
35	0.858	0.572	"	"	0.348	0.696
36	0.828	0.552	"	"	0.340	0.680
37	0.798	0.532	"	"	0.332	0.664
38	0.768	0.512	"	"	0.324	0.648
39	0.738	0.492	"	"	0.318	0.636
40	0.714	0.476	"	"	0.312	0.624
41	0.690	0.460	"	"	0.304	0.608
42	0.666	0.444	"	"	0.298	0.596
43	0.642	0.428	"	"	0.292	0.584
44	0.618	0.412	"	"	0.288	0.576
45	0.600	0.400	"	"	0.284	0.568

STRAINS IN TRUSSED BEAMS.

When a beam supported at the ends, is required to carry a greater load than its given capacity, and trussing is resorted to, it may become necessary to find what portion of the load is borne by the different members of the trussed beam.

Reference.

Let W = Load acting on truss at a supported point. (See figure.)

W_1 = That portion of W acting on diagonals.

W_2 = That portion of W acting on beam.

A_1 = Sectional area of diagonal.

A_2 = Sectional area of beam.

E_1 = Modulus of elasticity of material in diagonals.

E_2 = Modulus of elasticity of material in beam.

a = Length of diagonal.

b = Distance between center of beam and point of support.

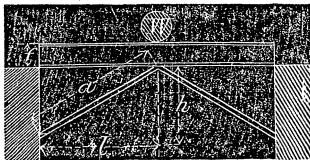
c = Distance between abutment and point of support.

f = Depth of beam.

h = Depth of truss.

l = Distance between center of beam and abutment.

[NOTE.—Use the same unit of length and weight.]

No. 1.*Fig. 209*

$$\frac{W_1}{W_2} = \frac{l^3}{a^3} \cdot \frac{h^2}{f^2} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{l^3}{a^3} \cdot \frac{h^2}{f^2} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2} W_2$$

$$W_2 = \frac{a^3}{l^3} \cdot \frac{f^2}{h^2} \cdot \frac{A_2}{A_1} \cdot \frac{E_2}{E_1} W_1$$

$$A_1 = \frac{W_1}{W_2} \cdot \frac{a^3}{l^3} \cdot \frac{f^2}{h^2} \cdot \frac{A_2}{E_1} \cdot \frac{E_2}{E_1}$$

$$A_2 = \frac{W_2}{W_1} \cdot \frac{l^3}{a^3} \cdot \frac{h^2 A_1}{f^2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{\frac{W_1}{W_2}}{\frac{W_1}{W_2} + 1} \cdot W \qquad W_2 = \frac{1}{\frac{W_1}{W_2} + 1}$$

When load is equally distributed W becomes $\frac{5}{8} W$.

No. 2.

Fig. 210.

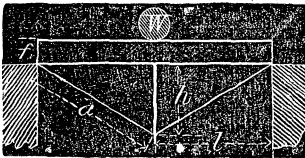
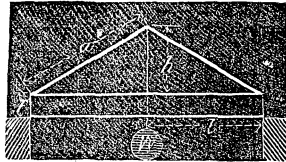


Fig. 211.



$$\frac{W_1}{W_2} = \frac{1}{2} \cdot \frac{l^3}{a^3} \cdot \frac{h^2}{f^2} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{W_2}{2} \cdot \frac{l^3}{a^3} \cdot \frac{h^2}{f^2} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2}$$

$$W_2 = 2 W_1 \cdot \frac{a^3}{l^3} \cdot \frac{f^2}{h^2} \cdot \frac{A_2}{A_1} \cdot \frac{E_2}{E_1}$$

$$A_1 = \frac{2 W_1}{W_2} \cdot \frac{a^3}{l^3} \cdot \frac{f^2 A_2}{h^2} \cdot \frac{E_2}{E_1}$$

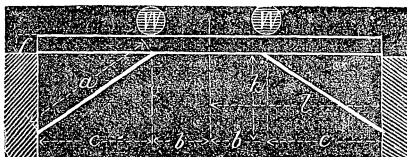
$$A_2 = A_1 \frac{W_2}{2 W_1} \cdot \frac{l^3}{a^3} \cdot \frac{h^2}{f^2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{\frac{W_1}{W_2}}{\frac{W_1}{W_2} + 1} \cdot W \qquad W_2 = \frac{W}{\frac{W_1}{W_2} + 1}$$

When load is equally distributed W becomes $\frac{5}{8} W$.

No. 3.

Fig. 212.



$$\frac{W_1}{W_2} = \frac{h^2}{f^2} \cdot \frac{(l^2 - b^2) c}{a(a^2 + bc)} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{h^2}{f^2} \cdot \frac{(l^2 - b^2) c}{a(a^2 + bc)} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2} \cdot W_2$$

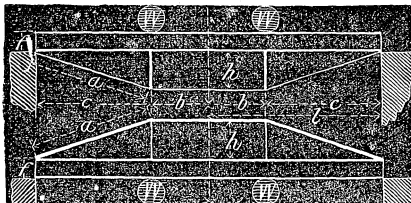
$$W_2 = \frac{f^2}{h^2} \cdot \frac{a(a^2 + bc)}{(l^2 - b^2) c} \cdot \frac{A_2}{A_1} \cdot \frac{E_2}{E_1} \cdot W_1$$

$$A_1 = \frac{W_1}{W_2} \cdot \frac{f^2}{h^2} \cdot \frac{a(a^2 + bc)}{(l^2 - b^2) c} \cdot \frac{A_2 \cdot E_2}{E_1}$$

$$A_2 = \frac{W_2}{W_1} \cdot \frac{h^2}{f^2} \cdot \frac{(l^2 - b^2) c}{a(a^2 + bc)} \cdot \frac{A_1 \cdot E_1}{E_2}$$

$$W_1 = \frac{\frac{W_1}{W_2}}{\frac{W_1}{W_2} + 1} \cdot W \qquad W_2 = \frac{W}{\frac{W_1}{W_2} + 1}$$

When load is equally distributed W becomes $\frac{3}{8} W$.

No. 4.*Figs. 213 and 214.*

$$\frac{W_1}{W_2} = \frac{h^2}{2f^2} \cdot \frac{(l^2 - b^2)c}{a(a^2 + bc)} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2}$$

$$W_1 = \frac{h^2}{2f^2} \cdot \frac{(l^2 - b^2)c}{a(a^2 + bc)} \cdot \frac{A_1}{A_2} \cdot \frac{E_1}{E_2} \cdot W$$

$$W_2 = 2W_1 \frac{f^2}{h^2} \cdot \frac{a(a^2 + bc)}{(l^2 - b^2)c} \cdot \frac{A_2}{A_1} \cdot \frac{E_2}{E_1}$$

$$A_1 = \frac{2W_1}{W_2} \cdot \frac{f^2}{h^2} \cdot \frac{a(a^2 + bc)}{(l^2 - b^2)c} \cdot \frac{A_2 \cdot E_2}{E_1}$$

$$A_2 = \frac{W_2}{2W_1} \cdot \frac{h^2}{f^2} \cdot \frac{(l^2 - b^2)c}{a(a^2 + bc)} \cdot \frac{A_1 \cdot E_1}{E_2}$$

$$W_1 = \frac{\frac{W_1}{W_2}}{\frac{W_1}{W_2} + 1} \cdot W \quad W_2 = \frac{W}{\frac{W_1}{W_2} + 1}$$

When load is equally distributed W becomes $\frac{2}{3} W$.

STRAINS IN TRUSSES, WITH PARALLEL BOOMS.

(Caused by Static and Moving Loads.)

The strain in the upper boom is always compressive.

The strain in the lower boom is always tensile.

All braces inclined *down* from the nearest abutment are in tension.

All braces inclined *up* from the nearest abutment are in compression.

The strains in the verticals and diagonals *increase* from the center of truss to abutment.

The strains in the booms *decrease* from the center of truss to abutment.

A moving load, advancing over a truss, &c., causes the maximum moment of rupture (which under an equally distributed load is at the center of truss) to shift to one side of the center, thereby changing the nature and amount of strain in web only. This requires either the enlargement of those members constituting the web or the addition of so-called counters, (braces, struts, or ties.)

To find the point from center of truss to where the addition of counters must commence, the following formula is used:

Let d = Distance from center of truss to point where maximum moment of rupture occurs, and where counter bracing must commence.

d' = Distance from nearest abutment to ditto.

$$\left. \begin{aligned} \text{Then will } d &= l \left[\frac{1}{2} + \frac{w}{w'} - \sqrt{\frac{w}{w'} \left(1 + \frac{w}{w'} \right)} \right] \\ \text{And } d' &= \frac{l}{2} - d = \frac{lw}{w'} \left[\left(\sqrt{1 + \frac{w'}{w}} \right) - 1 \right] \end{aligned} \right\}$$

These results will be found to agree with formulas for "Counter Strains" when V_m becomes negative.

Reference.

N = Total number of bays in a truss.

H_n = Horizontal strains in booms.

V_n = Strains in verticals.

Y_n = Strains in diagonals.

V_m = Vertical strains acting on counters Y_m .

Y_m = Strains in counters, opposite in kind to Y_n .

W = Weight of static load, equally distributed over whole length of truss.

W_1 = Weight of moving load, equally distributed over whole length of truss.

h = Height or depth of truss between the center of gravity of booms.

l = Span or length of truss from abutment to abutment.

n = Number of member, counting from abutment A .

m = Number of member, between center and abutment B .

r = Half the length of a panel or bay.

s = Length of a panel or bay.

w = Weight of static load per unit of length l .

w_1 = Weight of moving load per unit of length l .

v = Angle between horizontal and diagonal.

For other designations, see diagrams and examples.

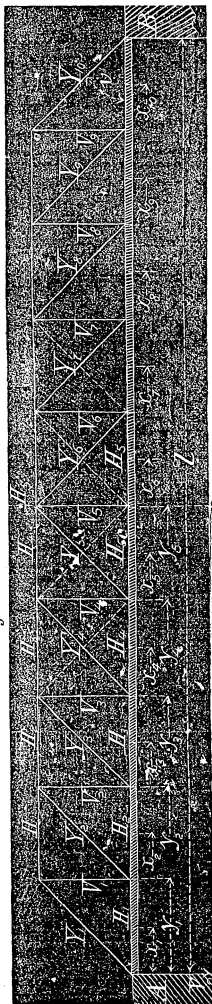
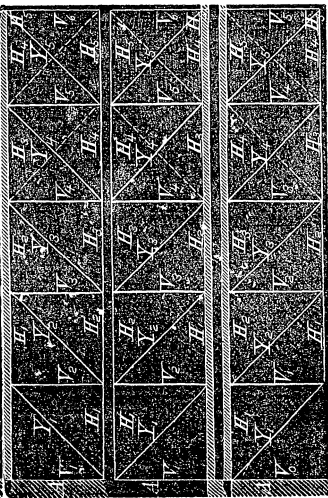
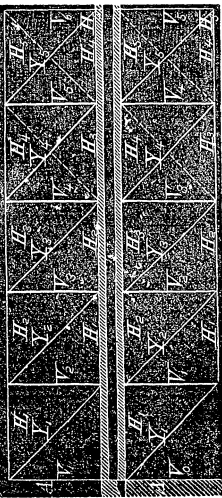
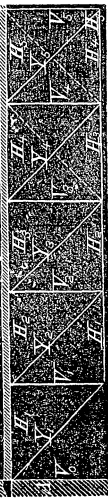
The angle v for Howe Truss is generally 45° .

The angle v for Whipple Truss is generally 45° .

The angle v for Lattice Truss is generally 45° .

The angle v for Warren Truss is generally 60° .

The proportion of height h to span l is from $\frac{1}{7}$ to $\frac{1}{15}$, generally $\frac{1}{10}$.

Fig. 215.—Lower boom loaded.*Fig. 216.—Upper boom loaded.**Fig. 217.—Lower boom loaded.**Fig. 218.—Upper boom loaded.*

HOWE TRUSS. (*Figs. 215, 216, 217, and 218.*)

Additional Reference.

x_n = Distance from abutment *A* to center of bay.

y_n = Distance from abutment *A* to apex of bay.

Static or Permanent Load, equally distributed over whole length of Truss.

Strains in Booms.

$$H_n = \frac{W}{2h} \cdot y_n - \frac{W}{2hl} \cdot y_n^2$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} x_n$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Moving and Static Load, each equally distributed per unit of length.

Strains in Booms.

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} x_n + \frac{W_1}{2l^2} (l - x_n)^2$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Strains in Counters.

$$V_m = \frac{W}{2} - \frac{W}{l} x_m + \frac{W_1}{2l^2} (l - x_m)^2 \quad Y_m = V_m \operatorname{cosec} v.$$

EXAMPLE. (Figs. 215, 216, 217, and 218.)

Moving Load, (as railway train passing over bridge.)

We will assume $W = 50,000$ lbs.

$W_1 = 100,000$ lbs.

$l = 100$ feet.

$h = 10$ feet.

$v = 45^\circ$, (cosec. $= 1.414$.)

Horizontal Strains in Booms, (compression in upper, tension in lower.)

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2 = \frac{50000 + 100000}{20} \cdot y_n - \frac{50000 + 100000}{2000} \cdot y_n^2$$

$$y_n - \frac{50000 + 100000}{2000} \cdot y_n^2 = 7500 \cdot y_n - 75 \cdot y_n^2$$

$$H_1 = 7500 \cdot 10 - 75 \cdot 100 = 67,500 \text{ lbs.}$$

$$H_2 = 7500 \cdot 20 - 75 \cdot 400 = 120,000 \text{ lbs.}$$

$$H_3 = 7500 \cdot 30 - 75 \cdot 900 = 157,500 \text{ lbs.}$$

$$H_4 = 7500 \cdot 40 - 75 \cdot 1600 = 180,000 \text{ lbs.}$$

$$H_5 = 7500 \cdot 50 - 75 \cdot 2500 = 187,500 \text{ lbs.}$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} \cdot x_n + \frac{W_1}{2l^2} \cdot (l - x_n)^2 = \frac{50000}{2} - \frac{50000}{100} \cdot x_n + \frac{100000}{20000} \cdot (l - x_n)^2$$

$$x_n + \frac{100000}{20000} \cdot (l - x_n)^2 = 25000 - 500 \cdot x_n + 5(l - x_n)^2$$

Strains in Figs. 215 216 217 218				
$V_1 = 25000 - 500 \cdot 5 + 5 \cdot 95^2 = 67625$	Ten.	Ten.	Com.	Com.
$V_2 = 25000 - 500 \cdot 15 + 5 \cdot 85^2 = 53625$	"	"	"	"
$V_3 = 25000 - 500 \cdot 25 + 5 \cdot 75^2 = 40625$	"	"	"	"
$V_4 = 25000 - 500 \cdot 35 + 5 \cdot 65^2 = 28625$	"	"	"	"
$V_5 = 25000 - 500 \cdot 45 + 5 \cdot 55^2 = 17625$	"	"	"	"

Counter Strains (V_m) for Strains in Counters.

$$V_6 = 25000 - 500 \cdot 55 + 5 \cdot 45^2 = 7625$$

$$V_7 = 25000 - 500 \cdot 65 + 5 \cdot 35^2 = 5625$$

Strains in Diagonals.

$$Y_n = V_n \text{ cosec. } v.$$

Strains in Figs. 215 216 217 218				
$Y_1 = 67625 \cdot 1.414 = 95,620$ lbs.	Com.	Com.	Ten.	Ten.
$Y_2 = 53625 \cdot 1.414 = 75,826$ lbs.	"	"	"	"
$Y_3 = 40625 \cdot 1.414 = 57,441$ lbs.	"	"	"	"
$Y_4 = 28625 \cdot 1.414 = 40,476$ lbs.	"	"	"	"
$Y_5 = 17625 \cdot 1.414 = 24,922$ lbs.	"	"	"	"

Strains in Counters, (dotted lines, Fig. 215, for example.)

$$Y_m = V_m \text{ cosec. } v.$$

Strains in Figs. 215 216 217 218				
$Y_6 = 7625 \cdot 1.414 = 10,782$ lbs.	Com.	Com.	Ten.	Ten.
$Y_7 = 5625 \cdot 1.414 = 7,954$ lbs.	"	"	"	"

Fig. 219.

LATTICE TRUSS WITH VERTICAL NUMBERS.

Fig. 219. Load on either Boom.

To compute the strains in this truss, the easiest method is to find the values of H_n , V_n , V_m , Y_n , and Y_m for a Howe Truss, (Figs. 215, 216, 217, and 218.) loaded in the same manner, (upper or lower boom.) These values in the following formulas for the above truss will give the required strains:

Strains in Booms. (S.)

$$S_1 = \frac{H_1}{2} \qquad S_4 = \frac{H_3 + H_4}{2}$$

$$S_2 = \frac{H_1 + H_2}{2} \qquad S_5 = \frac{H_4 + H_5}{2}$$

$$S_3 = \frac{H_2 + H_3}{2} \text{ Generally } S_n = \frac{H_n - 1 + H_n}{2}$$

Strains in Verticals. (U.)

Upper boom loaded—compression.

Lower boom loaded—tension.

$$U = \frac{W + W_1}{2N} \text{ constant.}$$

Strains in End Post (U_o.)

Upper boom loaded.

$$U_o = U + S_1 = \text{compression.}$$

Lower boom loaded.

$$U_o = S_1 = \text{compression.}$$

Strains in Diagonals. (D.)

$$D_1 = \frac{Y_1}{2} \qquad D_4 = \frac{Y_4}{2}$$

$$D_2 = \frac{Y_2}{2} \qquad D_5 = \frac{Y_5}{2}$$

$$D_3 = \frac{Y_3}{2} \qquad \text{Generally } D_n = \frac{Y_n}{2}$$

Strains in Counters.

$$\text{Generally } D_m = \frac{Y_m}{2}$$

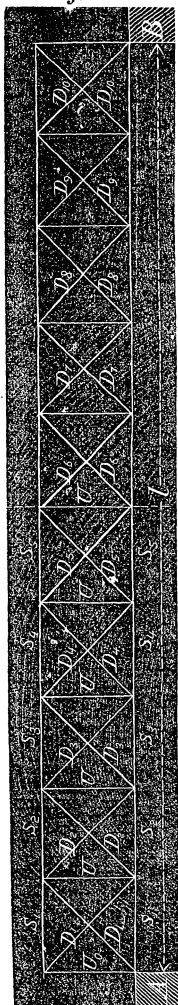
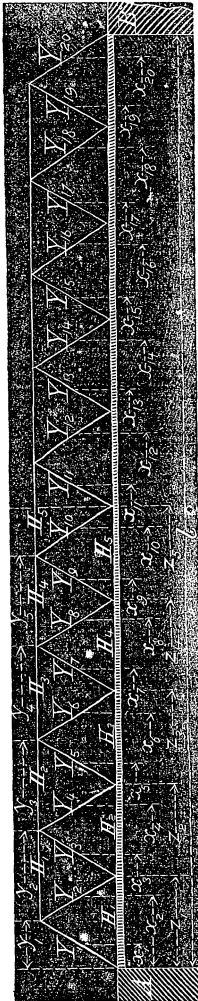


Fig. 220.



WARREN TRUSS.

Fig. 220. Lower Boom Loaded.

Additional Reference.

x_n = Distance from abutment *A* to center of diagonal.

y_n = Distance from abutment *A* to apex of bay of upper boom.

z_n = Distance from abutment *A* to apex of bay of lower boom.

Static or Permanent Load, equally distributed over whole length of Truss.

Strains in Booms.

Upper.

$$H_n = \frac{W}{2h} z_n - \frac{W}{2hl} \cdot z_n^2$$

Lower.

$$H_n = \frac{W}{2h} \cdot y_n - \frac{W}{2hl} \cdot y_n^2$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} x_n \quad (V_n \text{ acts at the end of } x_n.)$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Moving and Static Load, each equally distributed per unit of length.

Strains in Booms.

Upper.

$$H_n = \frac{W + W_1}{2h} \cdot z_n - \frac{W + W_1}{2hl} \cdot z_n^2$$

Lower.

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} x_n + \frac{W_1}{2l^2} (l - x_n)^2$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Strains in Counters.

$$V_m = \frac{W}{2} - \frac{W}{l}x_m + \frac{W_1}{2l^2}(l - x_m)^2 \quad Y_m = V_m \operatorname{cosec.} v.$$

EXAMPLE. (*Fig. 220.*)*Moving Load (as railway train passing over bridge) on lower Boom.*We will assume $W = 50,000$ lbs. $W_1 = 100,000$ lbs. $l = 100$ feet. $h = 10$ feet. $v = 63^\circ 20'$, ($\operatorname{cosec.} = 1.12$.)*Horizontal Strains in Upper Boom. (Compression.)*

$$H_n = \frac{W + W_1}{2h} \cdot z_n - \frac{W + W_1}{2hl} \cdot z_n^2 = \frac{50000 + 100000}{2.10}$$

$$z_n - \frac{50000 + 100000}{2.10 \cdot 100} \cdot z_n^2 = \frac{150000}{20} \cdot z_n -$$

$$\frac{150000}{2000} z_n^2 = 7500 \cdot z_n - 75 \cdot z_n^2$$

$$H_1 = 7500.10 - 75.100 = 67,500 \text{ lbs.}$$

$$H_2 = 7500.20 - 75.400 = 120,000 \text{ lbs.}$$

$$H_3 = 7500.30 - 75.900 = 157,500 \text{ lbs.}$$

$$H_4 = 7500.40 - 75.1600 = 180,000 \text{ lbs.}$$

$$H_5 = 7500.50 - 75.2500 = 187,500 \text{ lbs.}$$

Horizontal Strains in Lower Boom. (Tension.)

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2 = \frac{50000 + 100000}{2.10}$$

$$y_n - \frac{50000 + 100000}{2.10 \cdot 100} \cdot y_n^2 = \frac{150000}{20} \cdot y_n - \frac{150000}{2000} \cdot y_n^2$$

$$H_1 = 7500.5 - 75.25 = 37500 - 1875 = 35,625 \text{ lbs.}$$

$$H_2 = 7500.15 - 75.225 = 112500 - 16875 = 95,625 \text{ lbs.}$$

$$H_3 = 7500.25 - 75.625 = 187500 - 46875 = 140,625 \text{ lbs.}$$

$$H_4 = 7500.35 - 75.1225 = 262500 - 91875 = 170,625 \text{ lbs.}$$

$$H_5 = 7500.45 - 75.2025 = 337500 - 151875 = 185,625 \text{ lbs.}$$

Strains in Verticals.

$$Y_n = V_n \operatorname{cosec} v.$$

$$V_n = \frac{W}{2} - \frac{W}{l} \cdot x_n + \frac{W_1}{2l} \cdot (l - x_n) = \frac{50000}{2} - \frac{50000}{100} \cdot$$

$$x_n + \frac{100000}{2 \cdot 100^2} \cdot (100 - x_n)^2 = 25000 - 500x_n + 5 \cdot (100 - x_n)^2$$

$$V_1 = 25000 - 500 \cdot 2.5 + 5 \cdot 9506.25 = 71281.25.$$

$$V_2 = 25000 - 500 \cdot 7.5 + 5 \cdot 8556.25 = 64031.25.$$

$$V_3 = 25000 - 500 \cdot 12.5 + 5 \cdot 7656.25 = 57031.25.$$

$$V_4 = 25000 - 500 \cdot 17.5 + 5 \cdot 6806.25 = 50281.25.$$

$$V_5 = 25000 - 500 \cdot 22.5 + 5 \cdot 6006.25 = 43781.25.$$

$$V_6 = 25000 - 500 \cdot 27.5 + 5 \cdot 5256.25 = 37531.25.$$

$$V_7 = 25000 - 500 \cdot 32.5 + 5 \cdot 4556.25 = 31531.25.$$

$$V_8 = 25000 - 500 \cdot 37.5 + 5 \cdot 3906.25 = 25781.25.$$

$$V_9 = 25000 - 500 \cdot 42.5 + 5 \cdot 3306.25 = 20281.25.$$

$$V_{10} = 25000 - 500 \cdot 47.5 + 5 \cdot 2756.25 = 14031.25.$$

Counter Strains. (V_m .)

$$V_{11} = 25000 - 500 \cdot 52.5 + 5 \cdot 2256.25 = 10031.25.$$

$$V_{12} = 25000 - 500 \cdot 57.5 + 5 \cdot 1806.25 = 5281.25.$$

$$V_{13} = 25000 - 500 \cdot 62.5 + 5 \cdot 1406.25 = 781.25.$$

$$V_{14} = \text{Null}.$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

$$Y_1 = 71281.25 \cdot 1.12 = 79,835 \text{ lbs. Compression in } Y_1 \text{ and } Y_{20}.$$

$$Y_2 = 64031.25 \cdot 1.12 = 71,715 \text{ lbs. Tension in } Y_2 \text{ and } Y_{19}.$$

$$Y_3 = 57031.25 \cdot 1.12 = 63,875 \text{ lbs. Compression in } Y_3 \text{ and } Y_{18}.$$

$$Y_4 = 50281.25 \cdot 1.12 = 56,315 \text{ lbs. Tension in } Y_4 \text{ and } Y_{17}.$$

$$Y_5 = 43781.25 \cdot 1.12 = 49,035 \text{ lbs. Compression in } Y_5 \text{ and } Y_{16}.$$

$$Y_6 = 37531.25 \cdot 1.12 = 42,035 \text{ lbs. Tension in } Y_6 \text{ and } Y_{15}.$$

$$Y_7 = 31531.25 \cdot 1.12 = 35,315 \text{ lbs. Compression in } Y_7 \text{ and } Y_{14}.$$

$$Y_8 = 25781.25 \cdot 1.12 = 28,875 \text{ lbs. Tension in } Y_8 \text{ and } Y_{13}.$$

$$Y_9 = 20281.25 \cdot 1.12 = 22,715 \text{ lbs. Compression in } Y_9 \text{ and } Y_{12}.$$

$$Y_{10} = 14031.25 \cdot 1.12 = 15,715 \text{ lbs. Tension in } Y_{10} \text{ and } Y_{11}.$$

Counter Strains.

$$Y_m = V_m \operatorname{cosec} v.$$

$$Y_{11} = 10031.25 \cdot 1.12 = 11,235 \text{ lbs. Compression in } Y_{10} \text{ and } Y_{11}.$$

$$Y_{12} = 5281.25 \cdot 1.12 = 5,915 \text{ lbs. Tension in } Y_9 \text{ and } Y_{12}.$$

$$Y_{13} = 781.25 \cdot 1.12 = 875 \text{ lbs. Compression in } Y_8 \text{ and } Y_{13}.$$

Fig. 221.

WARREN TRUSS.

Fig. 221. Upper Boom Loaded.

Additional Reference.

x_n = Distance from abutment *A* to center of bay of upper boom.

y_n = Distance from abutment *A* to apex of bay of upper boom.

z_n = Distance from abutment *A* to apex of bay of lower boom.

Static or Permanent Load, equally distributed over whole length of Truss.

Strains in Booms.

Upper.

$$H_n = \frac{W}{2h} \cdot z_n - \left(\frac{W}{2hl} \cdot z_n^2 + \frac{Wl^2}{2hl} \right)$$

Lower.

$$H_n = \frac{W}{2h} \cdot y_n - \frac{W}{2hl} \cdot y_n^2$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} \cdot x_n$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Moving and Static Load, each equally distributed per unit of length.

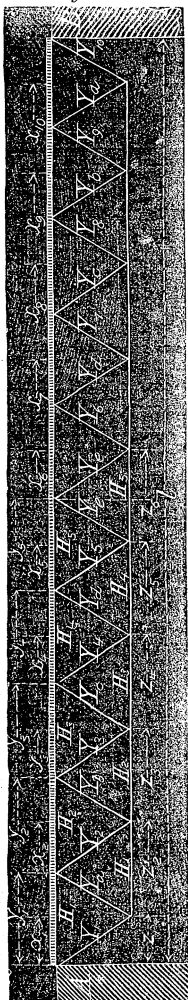
Strains in Booms.

Upper.

$$H_n = \frac{W + W_1}{2h} \cdot z_n - \left(\frac{W + W_1}{2hl} \cdot z_n^2 + \frac{(W + W_1) r^2}{2hl} \right)$$

Lower.

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2$$



Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} x_n + \frac{W_1}{2l^2} (l - x_n)^2$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec}. v.$$

Strains in Counters.

$$V_m = \frac{W}{2} - \frac{W}{l} x_m + \frac{W_1}{2l^2} (l - x_m)^2 \quad Y_m = V_m \operatorname{cosec}. v.$$

EXAMPLE. (*Fig. 221.*)

Moving Load (as railway train passing over bridge) on Upper Boom.

We will assume $W = 50,000$ lbs.

$W_1 = 100,000$ lbs.

$l = 100$ feet.

$h = 10$ feet.

$v = 63^\circ 20'$, $r = 5$ feet.

Horizontal Strains in Upper Boom. (Compression.)

$$\begin{aligned} H_n &= \frac{W + W_1}{2h} \cdot z_n - \left[\frac{W + W_1}{2hl} \cdot z_n^2 + \frac{(W + W_1)r^2}{2hl} \right] = \\ &= \frac{150000}{20} \cdot z_n - \left[\frac{150000}{2000} \cdot z_n^2 + \frac{150000 \cdot 5^2}{2000} \right] = \\ &= 7500 \cdot z_n - [75 \cdot z_n^2 + 1875] \end{aligned}$$

$$H_1 = 7500 \cdot 5 - [75 \cdot 25 + 1875] = 33,750 \text{ lbs.}$$

$$H_2 = 7500 \cdot 15 - [75 \cdot 225 + 1875] = 93,750 \text{ lbs.}$$

$$H_3 = 7500 \cdot 25 - [75 \cdot 625 + 1875] = 138,750 \text{ lbs.}$$

$$H_4 = 7500 \cdot 35 - [75 \cdot 1225 + 1875] = 168,750 \text{ lbs.}$$

$$H_5 = 7500 \cdot 45 - [75 \cdot 2025 + 1875] = 183,750 \text{ lbs.}$$

Horizontal Strains in Lower Boom (Tension.)

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2 = 7500 \cdot y_n - 75 \cdot y_n^2$$

$$H_1 = 7500 \cdot 10 - 75 \cdot 100 = 67,500 \text{ lbs.}$$

$$H_2 = 7500 \cdot 20 - 75 \cdot 400 = 120,000 \text{ lbs.}$$

$$H_3 = 7500 \cdot 30 - 75 \cdot 900 = 157,500 \text{ lbs.}$$

$$H_4 = 7500 \cdot 40 - 75 \cdot 1600 = 180,000 \text{ lbs.}$$

$$H_5 = 7500 \cdot 50 - 75 \cdot 2500 = 187,500 \text{ lbs.}$$

Strains in Verticals.

$$V_n = \frac{W}{2} - \frac{W}{l} \cdot x_n + \frac{W_1}{2l^2} (l - x_n)^2 = 25000 - 500 \cdot x_n + 5 \cdot (l - x_n)^2$$

$$V_1 = 25000 - 500 \cdot 5 + 5 \cdot 95^2 = 67,625 \text{ lbs.}$$

$$V_2 = 25000 - 500 \cdot 15 + 5 \cdot 85^2 = 53,625 \text{ lbs.}$$

$$V_3 = 25000 - 500 \cdot 25 + 5 \cdot 75^2 = 40,625 \text{ lbs.}$$

$$V_4 = 25000 - 500 \cdot 35 + 5 \cdot 65^2 = 28,625 \text{ lbs.}$$

$$V_5 = 25000 - 500 \cdot 45 + 5 \cdot 55^2 = 17,625 \text{ lbs.}$$

Counter Strains.

$$V_6 = 25000 - 500 \cdot 55 + 5 \cdot 45^2 = 7,625 \text{ lbs.}$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec.} v.$$

$$Y_1 = 67625 \cdot 1.12 = 75,740 \text{ lbs.} \quad \text{Tension in } Y_1 \text{ and } Y_{10};$$

compression in Y_a and Y_a .

$$Y_2 = 53625 \cdot 1.12 = 60,060 \text{ lbs.} \quad \text{Tension in } Y_2 \text{ and } Y_9;$$

compression in Y_b and Y_b .

$$Y_3 = 40625 \cdot 1.12 = 45,500 \text{ lbs.} \quad \text{Tension in } Y_3 \text{ and } Y_8;$$

compression in Y_c and Y_c .

$$Y_4 = 28625 \cdot 1.12 = 32,060 \text{ lbs.} \quad \text{Tension in } Y_4 \text{ and } Y_7;$$

compression in Y_d and Y_d .

$$Y_5 = 17625 \cdot 1.12 = 19,740 \text{ lbs.} \quad \text{Tension in } Y_5 \text{ and } Y_6;$$

compression in Y_e and Y_e .

Counter Strains.

$$Y_m = V_m \operatorname{cosec.} v.$$

$$Y_6 = 7625 \cdot 1.12 = 8,540 \text{ lbs.} \quad \text{Compression in } Y_5 \text{ and } Y_6;$$

tension in Y_e and Y_e .

Fig. 222.

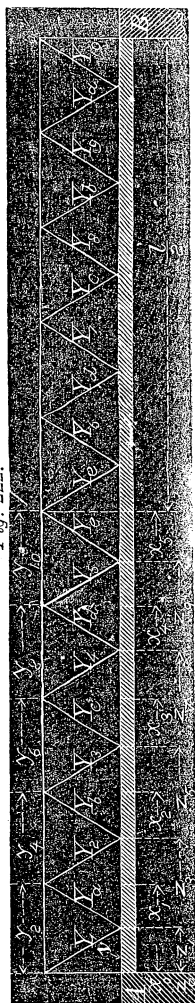


Fig. 223.

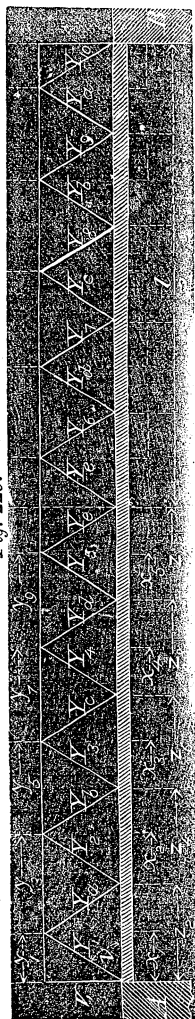
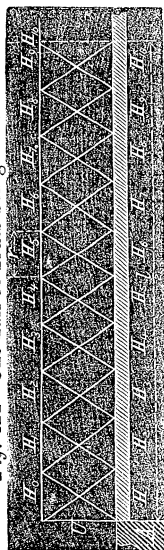


Fig. 224—One-half of Truss being shown.



LATTICE TRUSS. (*Figs. 222, 223, and 224.*)*Lower Boom Loaded.**Additional Reference.* r = Half the length of a bay of *simple* truss. (*Figs. 222 and 223.*) x_n = Distance from abutment *A* to center of bay of *lower* boom. y_n = Distance from abutment *A* to apex of bay of *upper* boom. z_n = Distance from abutment *A* to apex of bay of *lower* boom.

The formulas are for the strains in the simple trusses, (*Figs. 222 and 223.*) *Fig. 224* shows the simple trusses combined, constituting the Lattice Truss.

When the upper boom is loaded, treat the strains as acting upward and the truss inverted: the strains will be of the same amount in each member, but different in kind.

Static or Permanent Load, equally distributed over whole length of Truss.

Strains in Booms.

Upper.

$$H_n = \frac{W}{2h} \cdot \left(z_n + \frac{r}{2}\right) - \frac{W}{2hl} \cdot \left(z_n + \frac{r}{2}\right)^2 + \frac{Wr^2}{8hl}$$

Lower.

$$H_n = \frac{W}{2h} \cdot \left(y_n - \frac{r}{2}\right) - \frac{W}{2hl} \cdot \left(y_n - \frac{r}{2}\right)^2 - \frac{3Wr^2}{8hl}$$

Strains in Verticals.

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Moving and Static Load, each equally distributed per unit of length.

Strains in Booms.

Upper.

$$H_n = \frac{W + W_1}{2h} \cdot \left(z_n + \frac{r}{2}\right) - \frac{W + W_1}{2hl} \cdot \left(z_n + \frac{r}{2}\right)^2 + \frac{(W + W_1)r^2}{8hl}$$

Lower.

$$H_n = \frac{W + W_1}{2h} \cdot \left(y_n - \frac{r}{2}\right) - \frac{W + W_1}{2hl} \cdot \left(y_n - \frac{r}{2}\right)^2 - \frac{3(W + W_1)r^2}{8hl}$$

Strains in Verticals.

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n + \frac{W_1}{4l^2} \cdot (l - x_n)^2$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Strains in Counters.

$$V_m = \frac{W}{4} - \frac{W}{2l} \cdot x_m + \frac{W_1}{4l^2} \cdot (l - x_m)^2 \quad Y_m = V_m \operatorname{cosec} v.$$

[NOTE.—The strains in Y_a, b, c, \dots are equal in amount, but different in kind to the strains in $Y_1, 2, 3, \dots$.

EXAMPLE. (*Figs. 222, 223, and 224.*)

Moving Load (as railway train passing over bridge) on Lower Boom.

We will assume $W = 50,000$ lbs.

$W_1 = 100,000$ lbs.

$l = 100$ feet.

$h = 10$ feet.

$v = 63^\circ 20'$, ($\operatorname{cosec} v = 1.12$), $r = 5$ feet..

Horizontal Strains in Upper Boom. (Compression. Fig. 224.)

$$H_n = \frac{W + W_1}{2h} \left(z_n + \frac{r}{2} \right) - \frac{W + W_1}{2hl} \left(z_n + \frac{r}{2} \right)^2 +$$

$$\frac{(W + W_1)r^2}{8hl} = 7500(z_n + 2.5) - 75(z_n + 2.5)^2 + 468.75$$

$$H_0 = 7500 \cdot (0 + 2.5) - 75 \cdot (0 + 2.5)^2 + 468.75 = 18,750 \text{ lbs.}$$

$$H_1 = 7500 \cdot (5 + 2.5) - 75 \cdot (5 + 2.5)^2 + 468.75 = 52,500 \text{ lbs.}$$

$$H_2 = 7500 \cdot (10 + 2.5) - 75 \cdot (10 + 2.5)^2 + 468.75 = 82,500 \text{ lbs.}$$

$$H_3 = 7500 \cdot (15 + 2.5) - 75 \cdot (15 + 2.5)^2 + 468.75 = 108,750 \text{ lbs.}$$

$$H_4 = 7500 \cdot (20 + 2.5) - 75 \cdot (20 + 2.5)^2 + 468.75 = 131,250 \text{ lbs.}$$

$$H_5 = 7500 \cdot (25 + 2.5) - 75 \cdot (25 + 2.5)^2 + 468.75 = 150,000 \text{ lbs.}$$

$$H_6 = 7500 \cdot (30 + 2.5) - 75 \cdot (30 + 2.5)^2 + 468.75 = 165,000 \text{ lbs.}$$

$$H_7 = 7500 \cdot (35 + 2.5) - 75 \cdot (35 + 2.5)^2 + 468.75 = 176,250 \text{ lbs.}$$

$$H_8 = 7500 \cdot (40 + 2.5) - 75 \cdot (40 + 2.5)^2 + 468.75 = 183,750 \text{ lbs.}$$

$$H_9 = 7500 \cdot (45 + 2.5) - 75 \cdot (45 + 2.5)^2 + 468.75 = 187,500 \text{ lbs.}$$

Horizontal Strains in Lower Boom. (Tension. Fig. 224.)

$$H_n = \frac{W + W_1}{2h} \cdot \left(y_n - \frac{r}{2}\right) - \frac{W + W_1}{2hl} \cdot \left(y_n - \frac{r}{2}\right)^2 - \frac{3(W + W_1)r^2}{8hl} = 7500 \cdot (y_n - 2.5) - 75 \cdot (y_n - 2.5)^2 - 1406.25$$

$$\begin{aligned} H_1 &= 7500 \cdot (5 - 2.5) - 75 \cdot (5 - 2.5)^2 - 1406.25 = 16,875 \text{ lbs.} \\ H_2 &= 7500 \cdot (10 - 2.5) - 75 \cdot (10 - 2.5)^2 - 1406.25 = 50,625 \text{ lbs.} \\ H_3 &= 7500 \cdot (15 - 2.5) - 75 \cdot (15 - 2.5)^2 - 1406.25 = 80,625 \text{ lbs.} \\ H_4 &= 7500 \cdot (20 - 2.5) - 75 \cdot (20 - 2.5)^2 - 1406.25 = 106,875 \text{ lbs.} \\ H_5 &= 7500 \cdot (25 - 2.5) - 75 \cdot (25 - 2.5)^2 - 1406.25 = 129,375 \text{ lbs.} \\ H_6 &= 7500 \cdot (30 - 2.5) - 75 \cdot (30 - 2.5)^2 - 1406.25 = 148,125 \text{ lbs.} \\ H_7 &= 7500 \cdot (35 - 2.5) - 75 \cdot (35 - 2.5)^2 - 1406.25 = 163,125 \text{ lbs.} \\ H_8 &= 7500 \cdot (40 - 2.5) - 75 \cdot (40 - 2.5)^2 - 1406.25 = 174,375 \text{ lbs.} \\ H_9 &= 7500 \cdot (45 - 2.5) - 75 \cdot (45 - 2.5)^2 - 1406.25 = 181,875 \text{ lbs.} \\ H_{10} &= 7500 \cdot (50 - 2.5) - 75 \cdot (50 - 2.5)^2 - 1406.25 = 185,625 \text{ lbs.} \end{aligned}$$

SIMPLE TRUSS. (Fig. 222.)

Strains in Verticals. (V_n .)

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n + \frac{W_1}{4l^2} \cdot (l - x_n)^2 = 12500 - 250 \cdot x_n + 2.5 \cdot (l - x_n)^2$$

$$\begin{aligned} V_1 &= 12500 - 250 \cdot 0 + 2.5 \cdot 100^2 = 37,250 \text{ lbs. Com. in } U. \\ V_2 &= 12500 - 250 \cdot 10 + 2.5 \cdot 90^2 = 30,250 \text{ lbs.} \\ V_3 &= 12500 - 250 \cdot 20 + 2.5 \cdot 80^2 = 22,500 \text{ lbs.} \\ V_4 &= 12500 - 250 \cdot 30 + 2.5 \cdot 70^2 = 17,250 \text{ lbs.} \\ V_5 &= 12500 - 250 \cdot 40 + 2.5 \cdot 60^2 = 11,500 \text{ lbs.} \end{aligned}$$

Counter Strains. (V_m .)

$$\begin{aligned} V_6 &= 12500 - 250 \cdot 50 + 2.5 \cdot 50^2 = 6,250 \text{ lbs.} \\ V_7 &= 12500 - 250 \cdot 60 + 2.5 \cdot 40^2 = 1,500 \text{ lbs.} \end{aligned}$$

Strains in Diagonals.

$$Y_n = V_n \text{ cosec.}$$

$$Y_1 = 37250 \cdot 1.12 = 41,720 \text{ lbs. Tension in } Y_1 \text{ and } Y_{10}; \\ \text{compression in } Y_a \text{ and } Y_a.$$

$$Y_2 = 30250 \cdot 1.12 = 33,880 \text{ lbs. Tension in } Y_2 \text{ and } Y_9; \\ \text{compression in } Y_b \text{ and } Y_b.$$

$$\begin{aligned}
 Y_3 &= 22500 \cdot 1.12 = 25,200 \text{ lbs.} && \text{Tension in } Y_3 \text{ and } Y_8; \\
 &&& \text{compression in } Y_c \text{ and } Y_e. \\
 Y_4 &= 17250 \cdot 1.12 = 19,320 \text{ lbs.} && \text{Tension in } Y_4 \text{ and } Y_7; \\
 &&& \text{compression in } Y_d \text{ and } Y_a. \\
 Y_5 &= 11500 \cdot 1.12 = 12,880 \text{ lbs.} && \text{Tension in } Y_5 \text{ and } Y_6; \\
 &&& \text{compression in } Y_e \text{ and } Y_c.
 \end{aligned}$$

Counter Strains.

$$Y_m = V_m \operatorname{cosec}. v.$$

$$\begin{aligned}
 Y_6 &= 6250 \cdot 1.12 = 7,000 \text{ lbs.} && \text{Compression in } Y_5 \text{ and } Y_6; \\
 &&& \text{tension in } Y_e \text{ and } Y_c. \\
 Y_7 &= 1500 \cdot 1.12 = 1,680 \text{ lbs.} && \text{Compression in } Y_4 \text{ and } Y_7; \\
 &&& \text{tension in } Y_d \text{ and } Y_a.
 \end{aligned}$$

SIMPLE TRUSS. (*Fig. 223.*)

Strains in Verticals. (V_n .)

$$\begin{aligned}
 V_1 &= 12500 - 250 \cdot 5 + 2.5 \cdot 95^2 = 33812.5. \\
 V_2 &= 12500 - 250 \cdot 15 + 2.5 \cdot 85^2 = 26812.5. \\
 V_3 &= 12500 - 250 \cdot 25 + 2.5 \cdot 75^2 = 20312.5. \\
 V_4 &= 12500 - 250 \cdot 35 + 2.5 \cdot 65^2 = 14312.5. \\
 V_5 &= 12500 - 250 \cdot 45 + 2.5 \cdot 55^2 = 8812.5.
 \end{aligned}$$

Counter Strains. (V_m .)

$$V_6 = 12500 - 250 \cdot 55 + 2.5 \cdot 45^2 = 3812.$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec}. v.$$

$$\begin{aligned}
 Y_1 &= 33812.5 \cdot 1.12 = 37,870 \text{ lbs.} && \text{Compression in } Y_1 \text{ and } Y_{10}, \\
 &&& \text{tension in } Y_a \text{ and } Y_e. \\
 Y_2 &= 26812.5 \cdot 1.12 = 30,030 \text{ lbs.} && \text{Compression in } Y_2 \text{ and } Y_9; \\
 &&& \text{tension in } Y_b \text{ and } Y_d. \\
 Y_3 &= 20312.5 \cdot 1.12 = 22,750 \text{ lbs.} && \text{Compression in } Y_3 \text{ and } Y_8; \\
 &&& \text{tension in } Y_c \text{ and } Y_e. \\
 Y_4 &= 14312.5 \cdot 1.12 = 16,030 \text{ lbs.} && \text{Compression in } Y_4 \text{ and } Y_7; \\
 &&& \text{tension in } Y_d \text{ and } Y_a. \\
 Y_5 &= 8812.5 \cdot 1.12 = 9,870 \text{ lbs.} && \text{Compression in } Y_5 \text{ and } Y_6; \\
 &&& \text{tension in } Y_e \text{ and } Y_c.
 \end{aligned}$$

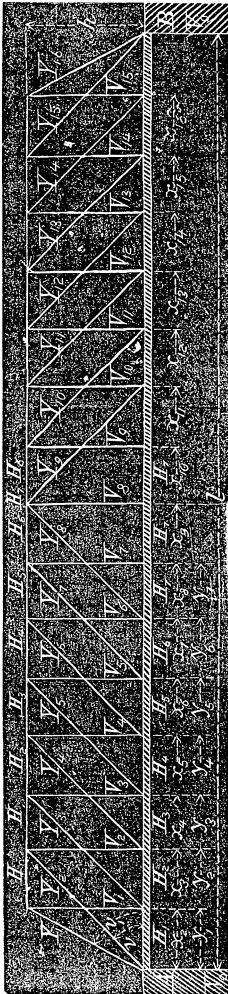
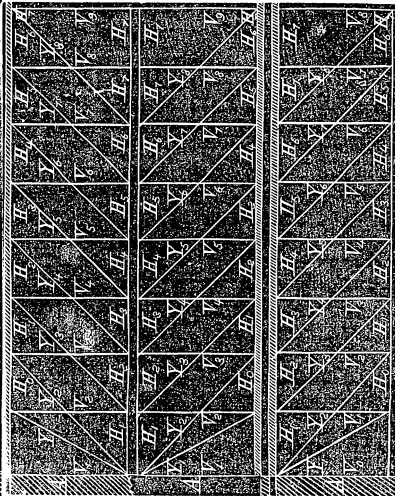
Counter Strains.

$$Y_m = V_m \operatorname{cosec}. v.$$

$$Y_6 = 3812.5 \cdot 1.12 = 4,270 \text{ lbs.} \quad \text{Tension in } Y_5 \text{ and } Y_6; \text{ compression in } Y_e \text{ and } Y_c.$$

Fig. 225.

Lower boom loaded.

*Fig. 226.*—Upper boom loaded.*Fig. 227.*—Lower boom loaded.*Fig. 228.*—Upper boom loaded.

WHIPPLE TRUSS. (*Figs. 225, 226, 227, and 228.*)

Additional Reference.

x_n, y_n = Distance from abutment *A* to end of bay.

$$x_1 = 0$$

Static or Permanent Load, equally distributed over whole length of Truss.

Strains in Booms.

$$H_n = \frac{W}{2h} \cdot y_n - \frac{W}{2hl} \cdot y_n^2 + \frac{sW}{2hl} \cdot y_n - \frac{sW}{4h}$$

Strains in Verticals.

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Moving and Static Load, each equally distributed per unit of length.

Strains in Booms.

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2 + \frac{s(W + W_1)}{2hl} \cdot y_n - \frac{s(W + W_1)}{4h}$$

Strains in Verticals.

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n + \frac{W_1}{4l^2} \cdot (l - x_n)^2$$

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Strains in Counters.

$$V_m = \frac{W}{4} - \frac{W}{2l} \cdot x_m + \frac{W_1}{4l^2} \cdot (l - x_m)^2 \quad Y_m = V_m \operatorname{cosec} v.$$

EXAMPLE. (Figs. 225, 226, 227, and 228.)

(With 20 Bays.)

*Moving Load, (as railway train passing over bridge.)*Let $W = 50,000$ lbs. $W_1 = 100,000$ lbs. $l = 100$ feet. $h = 10$ feet, $s = 5$ feet. $v = 45^\circ$. (End diagonals $v = 26^\circ 30'$.)*Horizontal Strains in Booms. (Compression in upper, tension in lower.)*

$$H_n = \frac{W + W_1}{2h} \cdot y_n - \frac{W + W_1}{2hl} \cdot y_n^2 + \frac{s(W + W_1)}{2hl} \cdot y_n - \frac{s(W + W_1)}{4h} = 7500 \cdot y_n - 75 \cdot y_n^2 - 375 \cdot y_n + 18750$$

$$H_0 = 7500 \cdot 0 - 75 \cdot 0^2 - 375 \cdot 0 + 18750 = 18,750 \text{ lbs.}$$

$$H_1 = 7500 \cdot 5 - 75 \cdot 5^2 - 375 \cdot 5 + 18750 = 52,500 \text{ lbs.}$$

$$H_2 = 7500 \cdot 10 - 75 \cdot 10^2 - 375 \cdot 10 + 18750 = 82,500 \text{ lbs.}$$

$$H_3 = 7500 \cdot 15 - 75 \cdot 15^2 - 375 \cdot 15 + 18750 = 108,750 \text{ lbs.}$$

$$H_4 = 7500 \cdot 20 - 75 \cdot 20^2 - 375 \cdot 20 + 18750 = 131,250 \text{ lbs.}$$

$$H_5 = 7500 \cdot 25 - 75 \cdot 25^2 - 375 \cdot 25 + 18750 = 150,000 \text{ lbs.}$$

$$H_6 = 7500 \cdot 30 - 75 \cdot 30^2 - 375 \cdot 30 + 18750 = 165,000 \text{ lbs.}$$

$$H_7 = 7500 \cdot 35 - 75 \cdot 35^2 - 375 \cdot 35 + 18750 = 176,250 \text{ lbs.}$$

$$H_8 = 7500 \cdot 40 - 75 \cdot 40^2 - 375 \cdot 40 + 18750 = 183,750 \text{ lbs.}$$

$$H_9 = 7500 \cdot 45 - 75 \cdot 45^2 - 375 \cdot 45 + 18750 = 187,500$$

$$= \left(\frac{(W + W_1)l}{8h} \right) \text{ lbs.}$$

Strains in Verticals.

$$V_n = \frac{W}{4} - \frac{W}{2l} \cdot x_n + \frac{W_1}{4l^2} \cdot (l - x_n)^2 = 12500 - 250 \cdot x_n + 2.5 \cdot (l - x_n)^2$$

$$V_0 = \frac{W + W_1}{2} = 75,000 \text{ lbs.}$$

Strains in Figs. 225 223 227 228

$$V_1 = 12500 - 250 \cdot 0 + 2.5 \cdot 100^2 = 37,500 \text{ lbs.} \quad \text{C. C. T. T.}$$

$$V_2 = 12500 - 250 \cdot 5 + 2.5 \cdot 95^2 = 33,812 \text{ lbs.} \quad \text{" " " "}$$

$$V_3 = 12500 - 250 \cdot 10 + 2.5 \cdot 90^2 = 30,250 \text{ lbs.} \quad \text{" " " "}$$

$$V_4 = 12500 - 250 \cdot 15 + 2.5 \cdot 85^2 = 26,812 \text{ lbs.} \quad \text{" " " "}$$

$$V_5 = 12500 - 250 \cdot 20 + 2.5 \cdot 80^2 = 23,500 \text{ lbs.} \quad \text{" " " "}$$

$$V_6 = 12500 - 250 \cdot 25 + 2.5 \cdot 75^2 = 20,312 \text{ lbs.} \quad \text{" " " "}$$

$$V_7 = 12500 - 250 \cdot 30 + 2.5 \cdot 70^2 = 17,250 \text{ lbs.} \quad \text{" " " "}$$

Strains in Figs. 225 226 227 228				
$V_8 = 12500 - 250 \cdot 35 + 2.5 \cdot 65^2 = 14,312$ lbs.	C.	C.	T.	T.
$V_9 = 12500 - 250 \cdot 40 + 2.5 \cdot 60^2 = 11,500$ lbs.	"	"	"	"
$V_{10} = 12500 - 250 \cdot 45 + 2.5 \cdot 55^2 = 8,812$ lbs.	"	"	"	"

V_m Acting on Counters.

$V_{11} = 12500 - 250 \cdot 50 + 2.5 \cdot 50^2 = 6,250$ lbs.
$V_{12} = 12500 - 250 \cdot 55 + 2.5 \cdot 45^2 = 3,812$ lbs.
$V_{13} = 12500 - 250 \cdot 60 + 2.5 \cdot 40^2 = 1,500$ lbs.

Strains in Diagonals.

$$Y_n = V_n \operatorname{cosec} v.$$

Strains in Figs. 225 226 227 228				
$Y_1 = 37500 \cdot 1.117 = 41,887$ lbs.	Ten.	Ten.	Com.	Com.
$Y_2 = 33812 \cdot 1.414 = 47,810$ lbs.	"	"	"	"
$Y_3 = 30250 \cdot 1.414 = 42,773$ lbs.	"	"	"	"
$Y_4 = 26812 \cdot 1.414 = 37,913$ lbs.	"	"	"	"
$Y_5 = 23500 \cdot 1.414 = 33,229$ lbs.	"	"	"	"
$Y_6 = 20312 \cdot 1.414 = 28,722$ lbs.	"	"	"	"
$Y_7 = 17250 \cdot 1.414 = 24,391$ lbs.	"	"	"	"
$Y_8 = 14312 \cdot 1.414 = 20,238$ lbs.	"	"	"	"
$Y_9 = 11500 \cdot 1.414 = 16,261$ lbs.	"	"	"	"
$Y_{10} = 8812 \cdot 1.414 = 12,461$ lbs.	"	"	"	"

Strains in Counters.

$Y_{11} = 6250 \cdot 1.414 = 8,837$ lbs.
$Y_{12} = 3812 \cdot 1.414 = 5,391$ lbs.
$Y_{13} = 1500 \cdot 1.414 = 2,121$ lbs.

[NOTE.—If counter braces are not inserted, V_{11} , V_{12} , and V_{13} , and Y_8 , Y_9 , and Y_{10} will have an additional strain, opposite in kind and equal to V_{11} , V_{12} , and V_{13} , and Y_{11} , Y_{12} , and Y_{13} ; but if counters are used, the strain V_{11} , V_{12} , and V_{13} will not occur in the structure, but will be necessary to determine the strain in Y_{11} , Y_{12} , and Y_{13} only. Y_{11} , Y_{12} , and Y_{13} will then be inclined in the same direction as the diagonals from abutment A to center of truss, the character of strain being the same. (See also "Howe Truss.")

Keep in mind that each half truss, as to the character and amount of strain in the respective members, is alike.]

STRAINS IN PARABOLIC CURVED TRUSSES — "BOW-STRING GIRDERS."

(Figs. 229, 230, 231, 232, 233, and 234.)

The strains in the lower boom (when horizontal) are the greatest, and equal in every bay, when the load is equally distributed over the whole length.

The strains in the arch or upper boom are also greatest when the load is equally distributed over the whole length; the strains gradually increasing from the middle to the supports.

The strains in the diagonals, whether single or double, in a bay are, when the load is equally distributed, everywhere *null*. When the load is unequally distributed, and one diagonal to each bay is used, they will be either in compression or tension. The character of the maximum of strains will be as follows: Assume the left half of truss to be loaded. All diagonals inclined *up* from left to right abutment are in tension; if inclined *down*, in compression. The character of strains will be *vice versa* when the right half only is loaded.

The strains in verticals are either compression, tension, or null. The maximum of compressive strain occurs when the diagonals in connection are under the greatest strain; that is, under an unequally distributed load. For other explanation, see diagram under variously-disposed loads.

In the following formulas and examples the diagonals (for a moving load) resist a tensional strain only, and the verticals a compressive. This would not be the case if one diagonal to each bay were used. In the latter case the diagonals and verticals would have to resist an alternate compressive and tensional strain.

When the trusses are inverted, the strains are different in kind, but not in amount.

Reference.

- A, B = Reaction of support.
- C = Compression in arch or upper boom.
- T = Tension in lower boom.
- D and H = Rise of arch.
- F and f = Vertical forces.
- W = Weight of moving and static load per unit of span or length.
- V = Strain in verticals.
- N = Total number of bays.
- a = Length of a bay.
- c = Length of a diagonal.
- d and h = Ordinates to parabola.
- l = Distance between supports or span.
- k = Total number of verticals = $N - 1$.
- m = Number of bays between support and V_n .

n = Number of a member, counting from support to middle of truss.

t = Tension in diagonal.

v and z = Angle between horizontal and member of polygon.

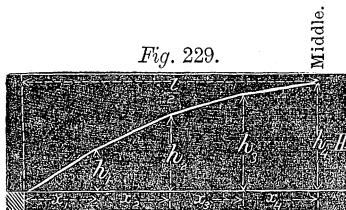
w = Weight of static load per unit of span or length.

w_j = Weight of moving load, equally distributed per unit of span or length.

u, x, y = Abscissas.

In the following diagrams, one-half of truss only is shown, the strains being alike in the respective members of each half:

Fig. 229.



Lower Boom Horizontal.

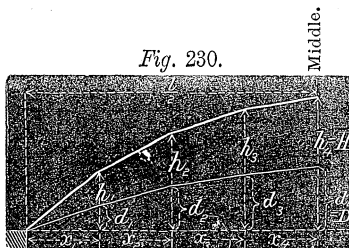
To find the ordinates h when H is given:

$$h_n = \frac{4Hx_n(l - x_n)}{l^2}$$

The value of T given, to find h :

$$h_n = -\frac{W(l - a)x_n}{2T} - \frac{1}{2}x_n^2 \frac{w}{T}$$

Fig. 230.



Lower Boom Curved.

To find the ordinates h or d when H or D is given:

$$h_n = \frac{4Hx_n(l - x_n)}{l^2}$$

$$d_n = \frac{4Dx_n(l - x_n)}{l^2}$$

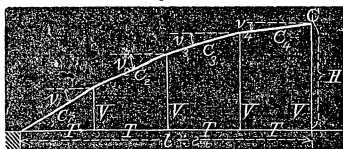
The value of T given, to find h :

$$h_n = \frac{W(l-a)x_n}{2T} - \frac{1}{2}x_n^2 \frac{w}{T}$$

Load equally distributed—Static Load. (Figs. 231 and 232.)

W = The weight of construction and applied load.

Fig. 231.



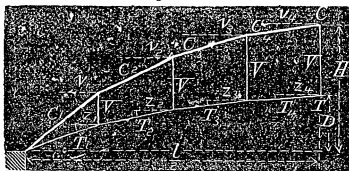
Lower Boom Loaded.

$$C_n = \frac{1}{8} \frac{Wl^2}{H} \sec. v_n \quad T = \frac{1}{8} \frac{Wl^2}{H} = C \quad V = \frac{wl}{N} = \text{tension.}$$

Upper Boom Loaded.

$$C'_n = \frac{1}{8} \frac{Wl^2}{H} \sec. v_n \quad T = \frac{1}{8} \frac{Wl^2}{H} = C \quad V = \text{null.}$$

Fig. 232.



Upper Boom Loaded. ($C = T$)

$$C_n = \frac{1}{8} \frac{Wl^2}{H-D} \sec. v_n \quad T_n = \frac{1}{8} \frac{Wl^2}{H-D} \sec. z_n$$

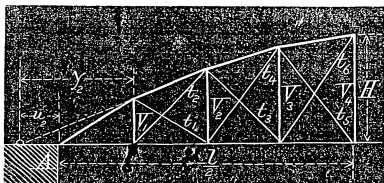
$$V = \frac{lw}{N} = \text{tension.}$$

Load unequally distributed—Moving Load. (Figs. 233 and 234.)

(Strains in Booms, same as for Static Load.)

Fig. 233.

$$y_n = h_n \cot. v_n; u_n = y_n - ma$$



Lower Boom Loaded.

$$t_n = \frac{w, l}{8H} c_n$$

$$V_n = F_n - f_n = \text{compression.}$$

$$F_n = B_n \left(\frac{u_n + Na}{u_n + ma} \right)$$

$$f_n = A_n \left(\frac{u_n}{u_n + ma} \right)$$

$$A_n = aw \left[\frac{(1+k-m)(k-m)}{2 \cdot N} \right]$$

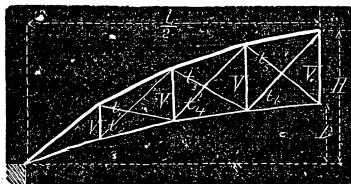
$$B_n = a(w + w_1) \left[\frac{(1+m)m}{2N} \right]$$

Upper Boom Loaded.

$$V_n = \frac{Wl}{8} = \text{compression.}$$

$$t_n = \frac{w, l}{8H} c_n$$

Fig. 234.



Upper Boom Loaded.

$$V_n = \frac{Wl}{8} = \text{compression.}$$

$$t_n = \frac{w, l}{8(H-D)} c_n$$

EXAMPLE. (Fig. 233.)

*Moving Load on Lower Boom.**Reference.*

$$\begin{array}{lll}
 l = 64 \text{ feet.} & c_1 = 8.7 \text{ feet.} & w = 125 \text{ lbs.} \\
 H = 8 \text{ feet.} & c_2 = c_3 = 10.0 \text{ feet.} & w_1 = 625 \text{ lbs.} \\
 a = 8 \text{ feet.} & c_4 = c_5 = 10.9 \text{ feet.} & W = w + w_1 = 750 \text{ lbs.} \\
 N = 8, k = 7. & c_6 = 11.3 \text{ feet.} &
 \end{array}$$

$$h_1 = \frac{4 \times 8 \times 8(64 - 8)}{64^2} = 3.5 \text{ feet.} \quad \begin{array}{l} u_1 = 8.0 - 8 = 0 \text{ feet.} \\ u_2 = 19.2 - 16 = 3.2 \text{ feet.} \end{array}$$

$$h_2 = \frac{4 \times 8 \times 16(64 - 16)}{64^2} = 6.0 \text{ feet.} \quad \begin{array}{l} u_3 = 40.0 - 24 = 16.0 \text{ feet.} \\ u_4 = 128.0 - 32 = 96.0 \text{ feet.} \end{array}$$

$$h_3 = \frac{4 \times 8 \times 24(64 - 24)}{64^2} = 7.5 \text{ feet.}$$

$$h_4 = H = 8.0 \text{ feet.}$$

$$\text{Tang. } v_1 = \frac{h_1}{a} = \frac{3.5}{8} = 23^\circ 37'.$$

$$\text{Tang. } v_2 = \frac{h_2 - h_1}{a} = \frac{6 - 3.5}{8} = 17^\circ 21'.$$

$$\text{Tang. } v_3 = \frac{h_3 - h_2}{a} = \frac{7.5 - 6}{8} = 10^\circ 38'.$$

$$\text{Tang. } v_4 = \frac{h_4 - h_3}{a} = \frac{8 - 7.5}{8} = 3^\circ 34' 30''.$$

$$\begin{array}{ll}
 y_1 = 3.5 \times 2.28 = 8.0 \text{ feet.} & y_3 = 7.5 \times 5.37 = 40.0 \text{ feet.} \\
 y_2 = 6.0 \times 3.20 = 19.2 \text{ feet.} & y_4 = 8.0 \times 16.00 = 128.0 \text{ feet.}
 \end{array}$$

$$T = C = \frac{1}{8} \frac{Wl^2}{H} = \frac{1}{8} \frac{750 \times 64^2}{8} = 48,000 \text{ lbs.}$$

$$C_n = C \sec. v_n.$$

$$\begin{array}{l}
 C_1 = 48000 \times 1.090 = 52,320 \text{ lbs.} \quad C_3 = 48000 \times 1.017 = 48,816 \text{ lbs.} \\
 C_2 = 48000 \times 1.047 = 50,256 \text{ lbs.} \quad C_4 = 48000 \times 1.0019 = 48,091 \text{ lbs.}
 \end{array}$$

$$t_1 = \frac{625 \times 64}{8 \times 8} \times 8.7 = 5437.5 \text{ lbs.}$$

$$t_2 = t_3 = \frac{625 \times 64}{8 \times 8} \times 10.0 = 6250.0 \text{ lbs.}$$

$$t_4 = t_5 = \frac{625 \times 64}{8 \times 8} \times 10.9 = 6802.5 \text{ lbs.}$$

$$t_6 = \frac{625 \times 64}{8 \times 8} \times 11.3 = 7062.5 \text{ lbs.}$$

$$A_1 = 8 \times 125 \left[\frac{(1+7-1)(7-1)}{2 \times 8} \right] = 2625$$

$$A_2 = 8 \times 125 \left[\frac{(1+7-2)(7-2)}{2 \times 8} \right] = 1875$$

$$A_3 = 8 \times 125 \left[\frac{(1+7-3)(7-3)}{2 \times 8} \right] = 1250$$

$$A_4 = 8 \times 125 \left[\frac{(1+7-4)(7-4)}{2 \times 8} \right] = 750$$

$$B_1 = 8(125 + 625) \left[\frac{(1+1)1}{2 \times 8} \right] = 750$$

$$B_2 = 8(125 + 625) \left[\frac{(1+2)2}{2 \times 8} \right] = 2250$$

$$B_3 = 8(125 + 625) \left[\frac{(1+3)3}{2 \times 8} \right] = 4500$$

$$B_4 = 8(125 + 625) \left[\frac{(1+4)4}{2 \times 8} \right] = 7500$$

$$F_1 = 750 \left(\frac{0 + 8 \times 8}{0 + 1 \times 8} \right) = 6000.0$$

$$F_2 = 2250 \left(\frac{3.2 + 8 \times 8}{3.2 + 2 \times 8} \right) = 7812.5$$

$$F_3 = 4500 \left(\frac{16 + 8 \times 8}{16 + 3 \times 8} \right) = 9000.0$$

$$F_4 = 7500 \left(\frac{96 + 8 \times 8}{96 + 4 \times 8} \right) = 9375.0$$

$$f_1 = 2625 \left(\frac{0}{0 + 1 \times 8} \right) = 0$$

$$f_2 = 1875 \left(\frac{3.2}{3.2 + 2 \times 8} \right) = 312.5$$

$$f_3 = 1250 \left(\frac{16}{16 + 3 \times 8} \right) = 500.0$$

$$f_4 = 750 \left(\frac{96}{96 + 4 \times 8} \right) = 562.5$$

$$V_1 = 6000 - 0 = 6,000 \text{ lbs. } V_3 = 9000 - 500 = 8,500 \text{ lbs.}$$

$$V_2 = 7812.5 - 312.5 = 7,500 \text{ lbs. } V_4 = 9375 - 562.5 = 8,812.5 \text{ lbs.}$$

CAPACITY AND STRENGTH OF PARABOLIC ARCHED BEAMS OR RIBS ORIGINALLY CURVED.

Reference. (All dimensions in inches.)

A = Sectional area of beam.

C = Compressive strain in direction of arch.

E = Modulus of elasticity.

H = Horizontal thrust at abutment, or tension on tie rod.

I = Moment of inertia of cross-section of beam.

R = Resistance of material to crushing, (to be divided by factor of safety.)

W = Concentrated load at crown of arch.

a = Vertical deflection at crown.

b = Horizontal deflection at abutments.

h = Rise of arch.

$2l$ = Distance between abutments = span.

s = Distance between neutral axis and farthest edge of section.

w = Load per unit of length, equally distributed horizontally.

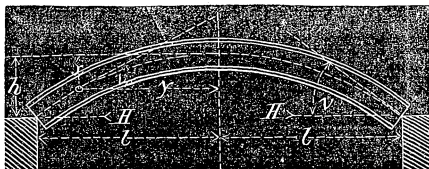
x = Vertical distance from crown to point of arch, intersected by y , say at O on diagram.

y = Horizontal distance from middle of arch to section where the amount of strain is desired.

v = Angle between horizontal and tangent to curve.

Horizontal Thrust, (resisted either by abutments or tie rod.)

Fig. 235. (All dimensions to line of pressure.)



To determine the curve or line of pressure:

$$\frac{x}{h} = \frac{y^2}{l^2} \quad \frac{y}{l} = \sqrt{\frac{x}{h}} \quad y = l \sqrt{\frac{x}{h}} \quad x = h \frac{y^2}{l^2}$$

$$\text{Tang. } v \text{ at any point} = \frac{2x}{y} = \frac{2\sqrt{hx}}{l}$$

$$\text{Tang. } v \text{ at abutment} = \frac{2h}{l}$$

Load concentrated at crown or middle of arch:

$$a = \frac{Wl_3}{256IE} \quad b = 0 \quad H = \frac{1}{2} W \left(\frac{25l}{32h} - \frac{h}{28l} \right)$$

$$C = \left(\frac{25l}{64h} - \frac{h}{56l} + \frac{hy}{l^2} - \frac{25hy^2}{32l^3} \right) W$$

$$R = \frac{25l}{64h} \frac{W}{A} + \frac{81 Wls}{1600I}$$

$$A = \frac{25l \times 1600I}{64h(R 1600I - 81 Wls)}$$

Load equally distributed:

$$a = 0 \quad b = 0 \quad H = \frac{wl^2}{2h} \quad C = \frac{wl^2}{2h} + \frac{wh y^2}{l^2}$$

$$R = \frac{C}{A} = \left(\frac{l^2}{2h} + \frac{h y^2}{l^2} \right) \frac{w}{A} \quad A = \frac{\left(\frac{l^2}{2h} + \frac{h y^2}{l^2} \right) w}{R}$$

STRAINS IN A POLYGONAL FRAME IN EQUILIBRIUM.

Load equally distributed over members of Frame.

Reference.

H = Horizontal strain in units of weight at foot.

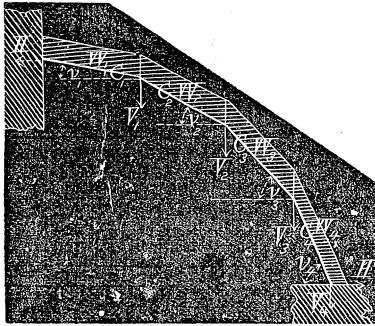
V_n = Vertical strain in units of weight at foot.

C_n = Compressive strain in units of weight in direction of member.

W_n = Load in units of weight, equally distributed over a member of the polygon.

v_n = Angle between horizontal and member.

Fig. 236.



$$H = \frac{1}{2} W \cot g. v_n \quad C_n = V_n \operatorname{cosec}. v_n$$

$$V_1 = \frac{1}{2} W_1$$

$$V_2 = V_1 + \frac{W_1 + W_2}{2} = \frac{W_1}{2} + \frac{W_1 + W_2}{2}$$

$$V_3 = V_2 + \frac{W_2 + W_3}{2} = \frac{W_1}{2} + \frac{W_1 + W_2}{2} + \frac{W_2 + W_3}{2}$$

$$V_4 = V_3 + \frac{W_3 + W_4}{2} = \frac{W_1}{2} + \frac{W_1 + W_2}{2} + \frac{W_2 + W_3}{2} + \frac{W_3 + W_4}{2} \dots \dots \&c.$$

For the equilibrium, v_1 being given :

$$\operatorname{Tang}. v_2 = \frac{V_1}{H} = \operatorname{tang}. v_1 + \frac{\frac{1}{2}(W_1 + W_2)}{H}$$

$$\operatorname{Tang}. v_3 = \frac{V_2}{H} = \operatorname{tang}. v_1 + \frac{\frac{1}{2}(W_1 + W_2) + \frac{1}{2}(W_2 + W_3)}{H}$$

$$\operatorname{Tang}. v_4 = \frac{V_3}{H} = \operatorname{tang}. v_1 +$$

$$\frac{\frac{1}{2}(W_1 + W_2) + \frac{1}{2}(W_2 + W_3) + \frac{1}{2}(W_3 + W_4)}{H}$$

The above can be used to compute the strains in ribs for dome construction.

STRAINS IN ROOF TRUSSES.

Reference. (Figs. 237 to 255.)

$$W = \left\{ \begin{array}{l} \text{Weight of construction.} \\ \text{Pressure of wind.} \\ \text{Pressure of snow.} \end{array} \right\} = \text{Load in units of weight, equally distributed over one rafter. (See Fig. 238.)}$$

C = Compression of member in units of weight.

T = Tension of member in units of weight.

L = Total span, or distance between abutments in units of length.

d, h, l , and S = Dimensions in units of length. (See Figures.)

v, y = Angles. (See Figures.)

The diagrams show only one-half of truss, (except Fig. 238,) the thick lines indicating compression, and the thin ones tension. (See "*Reaction of Supports*" for pressure on joints; also "*Compound Strains in Trussed Beams*.")

Compression in Rafters. (Trusses Nos. 1, 3, and 4.)

The compressive strain in the rafter gradually increases from ridge to abutments. Let x = Horizontal distance from abutment

to point where the strain is desired, and l half the span = $\frac{L}{2}$.

$$C \text{ for Truss No. 1} = W \sin. v \left(1 - \frac{x}{l}\right) + \frac{W}{2} \frac{\cos. v}{\text{tg. } v}$$

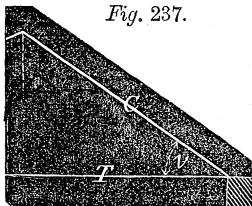
$$C \text{ for Truss No. 3} = W \sin. v \left(1 - \frac{x}{l}\right) + \frac{W}{2} \frac{\cos. v}{\text{tg. } (v + v_1)}$$

$$C \text{ for Truss No. 4} = W \sin. v \left(1 - \frac{x}{l}\right) + \frac{W}{2} \frac{\cos. v}{\text{tg. } (v - v_1)}$$

In the following examples the maximum of C is given:

Truss No. 1.

Fig. 237.



$$C = W \sin. v + \frac{W}{2} \frac{\cos. v}{\text{tg. } v}$$

$$T = \frac{W}{2} \cotg. v$$

EXAMPLE.

Let $W = 8,000$ lbs.
 $v = 26^\circ 30'$.

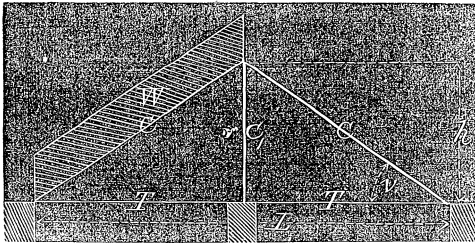
$$C = 8000 \times 0.44619 + \frac{8000}{2} \frac{0.89493}{0.49858} = 10,666 \text{ lbs. Com.}$$

$$\text{When } x = \frac{l}{2} \text{ then will } C = \frac{8000}{2 \times 0.44619} = 8,968 \text{ lbs. Com.}$$

$$T = \frac{8000}{2} 2.00 = 8,000 \text{ lbs. Tension.}$$

Truss No. 2.

Fig. 238.



$$C = \frac{W}{2} \sin. v \quad C_1 = W(\cos. v)^2$$

$$T = \frac{W}{2} \sin. v \cos. v = \frac{W}{4} \sin. 2v$$

EXAMPLE.

Let $W = 8,000$ lbs.
 $v = 26^\circ 30'$.

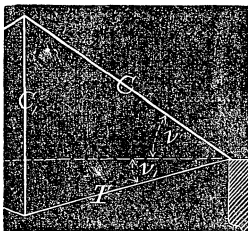
$$C = \frac{8000}{2} \times 0.4462 = 1,785 \text{ lbs. Compression.}$$

$$C_1 = 8000 \times 0.895^2 = 6,568 \text{ lbs. Compression.}$$

$$T = \frac{8000}{4} \times 0.7986 = 1,597 \text{ lbs. Tension.}$$

[NOTE.—When the rafters are fastened together at the ridge, they are under a cross-breaking strain only. Consequently there is no horizontal thrust at the abutments; that is, $T = 0$, and the compression in $C_1 = W$.]

Fig. 239.

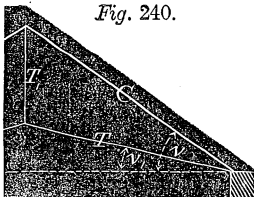
Truss No. 3.

$$C = W \sin. v + \frac{W}{2} \frac{\cos. v}{\operatorname{tg.}(v + v_1)}$$

$$C_1 = W \frac{\cos. v \sin. v_1}{\sin.(v + v_1)}$$

$$T = \frac{W}{2} \frac{\cos. v}{\sin.(v + v_1)}$$

Fig. 240.

Truss No. 4.

$$C = W \sin. v + \frac{W}{2} \frac{\cos. v}{\operatorname{tg.}(v - v_1)}$$

$$T = \frac{W}{2} \frac{\cos. v}{\sin.(v - v_1)}$$

$$T_1 = W \frac{\cos. v \sin. v_1}{\sin.(v - v_1)}$$

EXAMPLE.

Let $W = 8,000$ lbs.

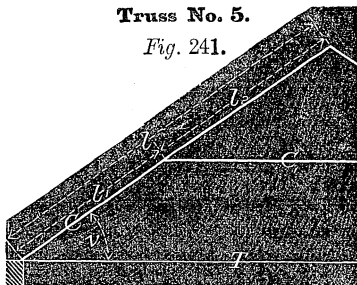
$v = 26^\circ 30'$.

$v_1 = 5^\circ 0'$.

$$C = 8000 \times 0.44619 + \frac{8000}{2} \frac{0.89493}{0.394} = 12,653 \text{ lbs. Com.}$$

$$T = \frac{8000}{2} \frac{0.894}{0.366} = 9,920 \text{ lbs. Tension.}$$

$$T_1 = 8000 \frac{0.894 \times 0.087}{0.366} = 1,720 \text{ lbs. Tension.}$$

Truss No. 5.*Fig. 241.*

$$C = \frac{13}{16} W \operatorname{cosec} v \quad C_1 = \frac{1}{2} W \cot g. v \quad T = \frac{1}{2} \left(1 + \frac{l_2}{l} \right) W \cot g. v$$

When there is no tie T , C_1 is under a tensile strain $= \frac{LW}{4h}$,
 h being the height from C_1 to ridge.

EXAMPLE.

Let $W = 8,000$ lbs.

$l = 22.36$ feet.

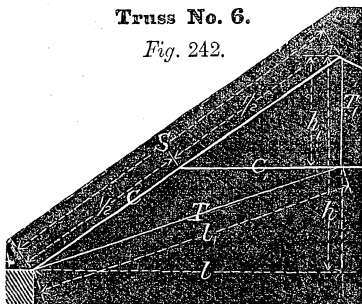
$l_1 = l_2 = 11.18$ feet.

$v = 26^\circ 30'$.

$$C = \frac{13}{16} 8000 \times 2.241 = 14,566 \text{ lbs. Compression.}$$

$$C_1 = \frac{1}{2} 8000 \times 2. = 8,000 \text{ lbs. Compression.}$$

$$T = \frac{1}{2} \left(1 + \frac{11.18}{22.36} \right) 8000 \times 2. = 12,000 \text{ lbs. Tension.}$$

Truss No. 6.*Fig. 242.*

$$C = \frac{WS^2 - \frac{13}{16} W(S^2 - hh_1)}{h_1 S}$$

$$T = \left(W - \frac{3}{16} W \right) \frac{l_1}{h_1}$$

$$C_1 = \frac{5}{8} W \frac{l}{h}$$

$$T_1 = 2(W - \frac{3}{16} W) \frac{h - h_1}{h_1}$$

EXAMPLE.

Let $W = 8,000$ lbs. $l = 20$ feet. $l_1 = 20.6$ feet. $h = 10$ feet. $h_1 = 5$ feet. $S = 22.36$.

$$C = \frac{8000 \times 500 - 1500(500 - 10 \times 5)}{5 \times 22.36} = 29,264 \text{ lbs. Com.}$$

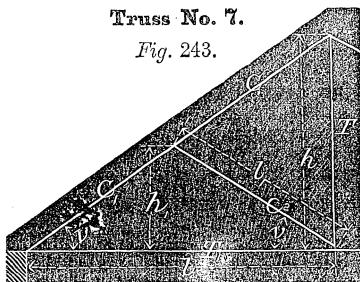
$$C_1 = 0.625 \times 8000 \frac{20}{10} = 10,000 \text{ lbs. Compression.}$$

$$T = (8000 - 1500) \frac{20.6}{5} = 26,780 \text{ lbs. Tension.}$$

$$T_1 = 2(8000 - 1500) \frac{10 - 5}{5} = 13,000 \text{ lbs. Tension.}$$

Truss No. 7.

Fig. 243.



$$C = W \frac{l}{2l \sin. v} = \frac{W}{2} \operatorname{cosec.} v \quad C_2 = \frac{5}{8} W \frac{l_1}{h} = \frac{5}{8} W \frac{\operatorname{cosec.} v_1}{2}$$

$$C_1 = \frac{1}{16} W \operatorname{cosec.} v \quad C_2 = \frac{5}{8} W \frac{\cos. v}{\sin. 2v}$$

$$T = \frac{5}{8} W \frac{h_1}{h} 2 = \frac{5}{8} W \quad T_1 = \frac{1}{16} W \cotg. v$$

EXAMPLE.

Let $W = 8,000$ lbs. $l = 20$ feet. $h = 10$ feet. $l_1 = 11.18$ feet. $v = 26^\circ 30'$. $v_1 = 26^\circ 30'$.

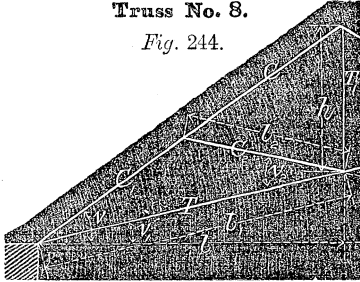
$$C = 8000 \frac{20}{2 \times 20 \times 0.44619} = 8,964 \text{ lbs. Compression.}$$

$$C_1 = 0.8125 \times 8000 \times 2.2411 = 14,567 \text{ lbs. Compression.}$$

$$C_2 = 0.625 \times 8000 \times 1.12 = 5,600 \text{ lbs. Compression.}$$

$$T = 0.625 \times 8000 = 5,000 \text{ lbs. Tension.}$$

$$T_1 = 0.8125 \times 8000 \times 2.0 = 13,000 \text{ lbs.}$$

Truss No. 8.*Fig. 244.*

$$C = \frac{T_1 + \frac{3}{8}W}{2 \sin. v} = W \frac{l}{2l_1 \sin.(v - v_1)} = \frac{W}{2} \frac{\cos. v_1}{\sin.(v - v_1)}$$

$$C_1 = \frac{13}{16} W \frac{\cos. v_1}{\sin.(v - v_1)}$$

$$C_2 = \frac{5}{8} W \frac{\cos. v}{\sin.(v - v_1 + v_2)} = \frac{5}{8} W \frac{l_2}{h}$$

$$T = \frac{13}{16} W \frac{\cos. v}{\sin.(v - v_1)}$$

$$T_1 = 2W \left[\frac{13}{16} \frac{\cos. v \sin. v_1}{\sin.(v - v_1)} + \frac{5}{8} \frac{\cos. v \sin.(v_2 - v_1)}{\sin.(v - v_1 + v_2)} \right] =$$

$$2 \left(T \sin. v_1 + C_2 \sin.(v_2 - v_1) \right) = W \frac{\sin. v \cos. v_1}{\sin.(v - v_1)} - \frac{3}{8} W$$

EXAMPLE.

Let $W = 8,000$ lbs.

$v = 26^\circ 30'.$

$v_1 = 9^\circ 20'.$

$v_2 = 19^\circ 0'.$

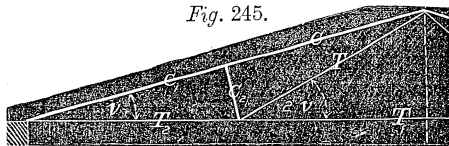
$$C = \frac{9000 + 0.375 \times 8000}{0.892} = 13,452 \text{ lbs. Compression.}$$

$$C_1 = 0.812 \times 8000 \frac{0.986}{0.295} = 21,710 \text{ lbs. Compression.}$$

$$C_2 = 0.625 \times 8000 \frac{0.895}{0.590} = 7,585 \text{ lbs. Compression.}$$

$$T = 0.812 \times 8000 \frac{0.895}{0.295} = 19,702 \text{ lbs. Tension.}$$

$$T_1 = 2 \times 8000 \left[0.812 \frac{0.812 \times 0.162}{0.295} + 0.625 \times \frac{0.895 \times 0.163}{0.590} \right] = 9,000 \text{ lbs. Tension.}$$

Truss No. 9.*Fig. 245.*

$$C = \frac{13}{16} W \frac{1}{\sin. v} - \frac{5}{8} W \sin. v$$

$$C_1 = \frac{13}{16} W \frac{1}{\sin. v} = \frac{13}{16} W \operatorname{cosec.} v$$

$$C_2 = \frac{5}{8} W \cos. v$$

$$T = \frac{5}{16} W \cotg. v$$

$$T_1 = \frac{13}{16} W \cotg. v - \frac{5}{16} W \cotg. v = \frac{1}{2} W \cotg. v$$

$$T_2 = \frac{13}{16} W \cotg. v$$

EXAMPLE.

Let $W = 8,000$ lbs.

$v = 26^\circ 30'.$

$$C = 0.812 \times 8000 \times 2.241 - 0.625 \times 8000 \times 0.446 = 12,336 \text{ lbs. Compression.}$$

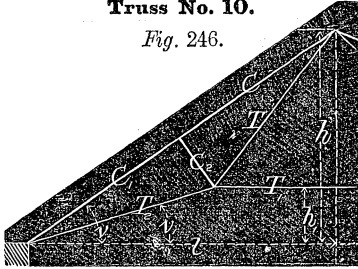
$$C_1 = 0.812 \times 8000 \times 2.241 = 14,566 \text{ lbs. Compression.}$$

$$C_2 = 0.625 \times 8000 \times 0.895 = 4,475 \text{ lbs. Compression.}$$

$$T = 0.312 \times 8000 \times 2. = 4,992 \text{ lbs. Tension.}$$

$$T_1 = 0.812 \times 8000 \times 2 - 0.312 \times 8000 \times 2. = 8,000 \text{ lbs. Tension.}$$

$$T_2 = 0.812 \times 8000 \times 2. = 12,992 \text{ lbs. Tension.}$$

Truss No. 10.*Fig. 246.*

$$C = \frac{1\frac{3}{8}}{16} W \frac{\cos. v_1}{\sin. (v - v_1)} - \frac{5}{8} W \sin. v$$

$$C_1 = \frac{1\frac{3}{8}}{16} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$C_2 = \frac{5}{8} W \cos. v.$$

$$T = \frac{1}{\sin. (2v - v_1)} \left[\frac{1\frac{3}{8}}{16} W \frac{\cos. v \sin. v_1}{\sin. (v - v_1)} + \frac{5}{8} W \cos.^2 v \right]$$

$$T_1 = \frac{1\frac{3}{8}}{16} W \frac{\cos. v \cos. v_1}{\sin. (v - v_1)} - T \cos. (2v - v_1) - \frac{5}{8} W \sin. \cos. v$$

$$= \frac{W}{2} \frac{l}{h - h_1}$$

$$T_2 = \frac{1\frac{3}{8}}{16} W \frac{\cos. v}{\sin. (v - v_1)}$$

EXAMPLE.Let $W = 8,000$ lbs. $v_1 = 9^\circ 20'$. $h = 10$ feet. $v = 26^\circ 30'$. $l = 20$ feet. $h_1 = 2$ feet.

$$C = 0.8125 \times 8000 \frac{0.987}{0.295} - 0.625 \times 8000 \times 0.446 = 19,517 \text{ lbs.}$$

Compression.

$$C_1 = 0.8125 \times 8000 \frac{0.987}{0.295} = 21,747 \text{ lbs. Compression.}$$

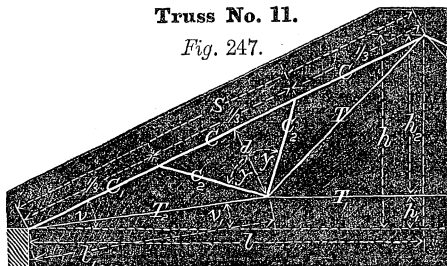
$$C_2 = 0.625 \times 8000 \times 0.895 = 4,475 \text{ lbs. Compression.}$$

$$T = \frac{1}{0.6905} \left(0.8125 \times 8000 \frac{0.895 \times 0.162}{0.295} + 0.625 \times 8000 \times \right.$$

$$\left. 0.895^2 \right) = 7,163 \text{ lbs. Tension.}$$

$$T_1 = \frac{8000}{2} \times \frac{20}{10-2} = 10,000 \text{ lbs. Tension.}$$

$$T_2 = 0.8125 \times 8000 \frac{0.895}{0.295} = 19,720 \text{ lbs. Tension.}$$

Truss No. 11.*Fig. 247.*

$$C = \frac{1\frac{3}{8}}{1\frac{6}} W \frac{\cos. v_1}{\sin. (v - v_1)} - \frac{5}{8} W \sin. v$$

$$C_1 = \frac{1\frac{3}{8}}{1\frac{6}} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$C_2 = \frac{1\frac{1}{8}}{3\frac{0}{0}} W \frac{\cos. v}{\cos. y}$$

$$T = \frac{1}{8} W \frac{\cos. v}{\sin. (2v - v_1)} \left(\frac{1\frac{3}{8}}{2} \frac{\sin. v_1}{\sin. (v - v_1)} + 5 \cos. v \right)$$

$$T_1 = \frac{W}{2} \frac{l}{h - h_1}$$

$$T_2 = \frac{1\frac{3}{8}}{1\frac{6}} W \frac{\cos. v}{\sin. (v - v_1)}$$

EXAMPLE.

Let $W = 8,000$ lbs. $y = 50^\circ$. $h = 10$ feet. $l = 20$ feet.
 $v = 26^\circ 30'$. $v_1 = 9^\circ 20'$. $h_1 = 2$ feet. $S = 22.36$ feet.

$$C = 0.8125 \times 8000 \frac{0.981}{0.295} - 0.625 \times 8000 \times 0.446 = 19,517 \text{ lbs.}$$

Compression.

$$C_1 = 0.8125 \times 8000 \frac{0.987}{0.295} = 21,747 \text{ lbs. Compression.}$$

$$C_2 = 0.366 \times 8000 \frac{0.894}{0.642} = 4,070 \text{ lbs. Compression.}$$

$$T = 0.125 \times 8000 \frac{0.894}{0.690} \left(6.5 \frac{0.162}{0.295} + 5 \cdot 0.894 \right) = 11,050 \text{ lbs.}$$

Tension.

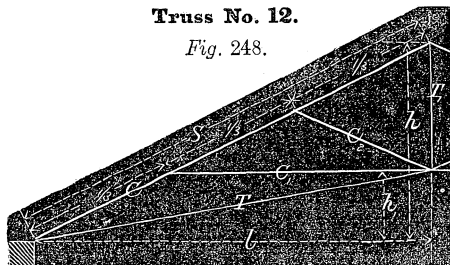
$$T_1 = 19486 \times 0.986 - 7421 \times 0.723 - 4930 \times 0.446 = 10,000 \text{ lbs.}$$

Tension.

$$T_2 = 0.812 \times 8000 \frac{0.894}{0.295} = 19,486 \text{ lbs. Tension.}$$

Truss No. 12.

Fig. 248.



$$C = \frac{43h^2 + 39l^2}{30 \times h \times l} W$$

$$T = \frac{13}{2} \frac{W \sqrt{h^2 + 9l^2}}{h}$$

$$C_1 = \frac{11}{30} W \frac{l}{h}$$

$$T_1 = \frac{37}{30} W$$

$$C_2 = \frac{11}{30} \frac{W}{2} \frac{S}{h}$$

EXAMPLE.Let $W = 8000$ lbs. $h = 10$ feet. $l = 20$ feet. $S = 22.36$ feet.

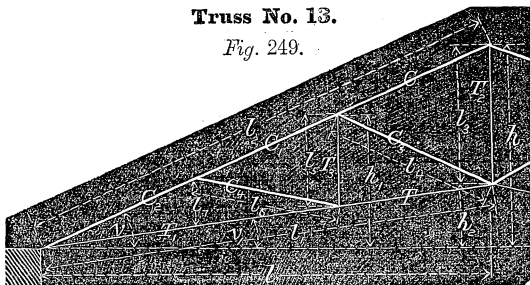
$$C = \frac{43 \times 100 \times 15600}{30 \times 10 \times 22.36} 8000 = 23,704 \text{ lbs. Compression.}$$

$$C_1 = 0.366 \times 8000 \frac{20}{10} = 5,856 \text{ lbs. Compression.}$$

$$C_2 = 0.366 \times \frac{8000}{2} \frac{22.36}{10} = 3,280 \text{ lbs. Compression.}$$

$$T = 0.866 \times \frac{8000}{2} \frac{\sqrt{100 + 3600}}{10} = 20,992 \text{ lbs. Tension.}$$

$$T_1 = 1.23 \times 8000 = 9,840 \text{ lbs. Tension.}$$

Truss No. 13.*Fig. 249.*

$$C = \frac{1}{2} W \frac{l_2}{l_3}$$

$$T = \frac{41}{60} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$C_1 = \frac{41}{60} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$T_1 = \frac{13}{15} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$C_2 = \frac{13}{15} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$T_2 = \frac{Wh}{l_3} - \frac{4}{15} W$$

$$C_3 = \frac{11}{20} W \frac{l_4}{l_3}$$

$$T_3 = \frac{11}{60} W$$

$$C_4 = \frac{11}{20} \times W \frac{l_6}{l_3}$$

EXAMPLE.

Let $W = 20,000$ lbs. $h = 20$ feet. $v = 21^\circ 40'$
 $l = 50$ feet. $l_2 = 53.8$ feet. $v = 0^\circ$

$$C = 0.5 \times 20000 \frac{53.8}{20} = 26,900 \text{ lbs. Compression.}$$

$$C_1 = 0.683 \times 20000 \frac{1}{0.369} = 37,018 \text{ lbs. Compression.}$$

$$C_2 = 0.866 \times 20000 \frac{1}{0.369} = 46,937 \text{ lbs. Compression.}$$

$$C_3 = 0.55 \times 20000 \frac{21.4}{20} = 11,770 \text{ lbs. Compression.}$$

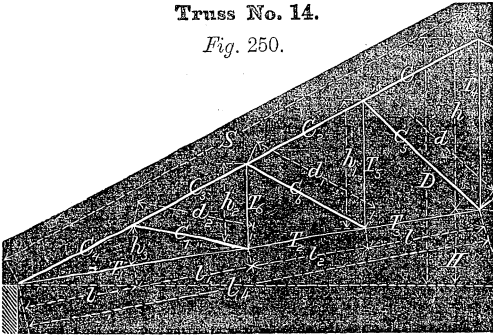
$$C_4 = 0.55 \times 20000 \frac{18}{20} = 9,900 \text{ lbs. Compression.}$$

$$T = 0.683 \times 20000 \times 2.517 = 34,382 \text{ lbs. Tension.}$$

$$T_1 = 0.866 \times 20000 \times 2.517 = 43,594 \text{ lbs. Tension.}$$

$$T_2 = \frac{20000 \times 20}{20} - 5333.33 = 14,666 \text{ lbs. Tension.}$$

$$T_3 = 0.183 \times 20000 = 3,660 \text{ lbs. Tension.}$$

Truss No. 14.*Fig. 250.*

$$C_1 = \frac{1}{2} W \frac{S}{h}$$

$$T_1 = (W - \frac{1}{10} W) \frac{l_4}{h}$$

$$C_2 = C_3 - \frac{1}{10} W \frac{S}{2h}$$

$$T_2 = T_1 - \frac{2}{7} W \times \frac{l_1}{h_2}$$

$$C_3 = C_4 - \frac{2}{7} W \frac{S}{2h}$$

$$T_3 = T_2 - C_6 \frac{l_2}{d_1}$$

$$C_4 = \frac{9}{70} W \frac{S}{h}$$

$$T_4 = \frac{WD}{h} - \frac{1}{5} W \frac{H}{h}$$

$$C_5 = (T_5 + \frac{2}{7} W) \frac{d}{h}$$

$$T_5 = C_6 \frac{h_2}{d_1}$$

$$C_6 = (T_6 + \frac{1}{70} W) \frac{d_1}{h_1}$$

$$T_6 = C_7 \frac{h_3}{d_2} = \frac{2}{7} W \frac{h_3}{h_2}$$

$$C_7 = \frac{2}{7} W \frac{d_2}{h_2}$$

EXAMPLE.

Let $W = 24,000$ lbs. Span = 100 feet $l = l_1 = l_2 = l_3 = 1.25$ feet.
 $h = 20$ feet. $H = 0$. $S = 53.85$ feet.

$$C_1 = 12000 \times \frac{53.85}{20} = 32,310 \text{ lbs. Compression.}$$

$$C_2 = 49088 - 0.228 \times 24000 \frac{53.85}{2 \times 20} = 41,728 \text{ lbs. Com.}$$

$$C_3 = 58320 - 0.286 \times 24000 \frac{53.85}{2 \times 20} = 49,088 \text{ lbs. Com.}$$

$$C_4 = 21600 \frac{53.85}{20} = 58,320 \text{ lbs. Compression.}$$

$$C_5 = (5801 + 0.286 \times 24000) \frac{19.5}{20} = 12,493 \text{ lbs. Com.}$$

$$C_6 = 3432 + 5484 \frac{16}{15} = 9,282 \text{ lbs. Compression.}$$

$$C_7 = 0.286 \times 24000 \frac{13.47}{10} = 9,245 \text{ lbs. Compression.}$$

$$T_1 = (24000 - 0.1 \times 24000) \frac{50}{20} = 54,000 \text{ lbs. Tension.}$$

$$T_2 = 54000 - 0.286 \times 24000 \frac{12.5}{10} = 45,420 \text{ lbs. Tension.}$$

$$T_3 = 45120 - 9282 \frac{12.5}{16} = 38,170 \text{ lbs. Tension.}$$

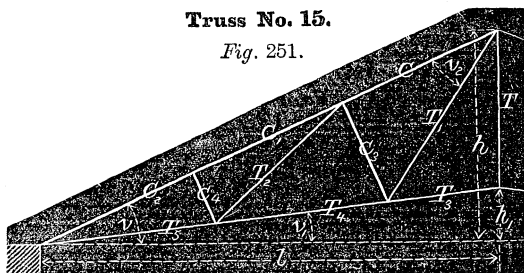
$$T_4 = 24000 - \frac{1}{5} 24000 = 19,200 \text{ lbs. Tension.}$$

$$T_5 = 9282 \frac{10}{16} = 5,801 \text{ lbs. Tension.}$$

$$T_6 = 0.286 \times 24000 \frac{5}{10} = 3,432 \text{ lbs. Tension.}$$

Truss No. 15.

Fig. 251.



$$C = \frac{13}{15} W \frac{\cos. v_1}{\sin. (v - v_1)} - \frac{11}{15} W \sin. v - \frac{11}{60} W \cos. v \cotg. (v - v_1)$$

$$C_1 = \frac{13}{15} W \frac{\cos. v_1}{\sin. (v - v_1)} - \frac{11}{30} W \sin. v \quad T_2 = \frac{11}{60} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$C_2 = \frac{13}{15} W \frac{\cos. v_1}{\sin. (v - v_1)} \quad T_3 = \frac{W}{2} \frac{l}{(h - h_1) \cos. v_1}$$

$$C_3 = \frac{11}{30} W \cos. v \quad T_4 = \frac{41}{60} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$C_4 = \frac{11}{30} W \cos. v \quad T_5 = \frac{13}{15} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$T = W \frac{l}{(h - h_1)} \tan g. v_1$$

$$T_1 = \frac{(T_4 - T_3) \cos. (v - v_1)}{\cos. v_2}$$

EXAMPLE.

$$\begin{array}{lll} \text{Let } W = 20,000 \text{ lbs.} & h = 20 \text{ feet.} & v_1 = 0. \\ l = 50 \text{ feet.} & v = 21^\circ 40'. & v_2 = 46^\circ 30'. \end{array}$$

$$C = 0.866 \times 20000 \frac{1}{0.369} - 0.733 \times 20000 \times 0.369 - 0.183 \times 20000 \times 0.929 \times 2.517 = 32,959 \text{ lbs. Compression.}$$

$$C_1 = 0.866 \times 20000 \times \frac{1}{0.369} - 0.366 \times 20000 \times 0.369 = 44,236 \text{ lbs. Compression.}$$

$$C_2 = 0.866 \times 20000 \times \frac{1}{0.369} = 46,937 \text{ lbs. Compression.}$$

$$C_3 = 0.55 \times 20000 \times 0.929 = 10,219 \text{ lbs. Compression.}$$

$$C_4 = 0.366 \times 20000 \times 0.929 = 6,800 \text{ lbs. Compression.}$$

$$T = 20000 \times \frac{40}{20} \times \tan g. v = \text{Null.}$$

$$T_1 = \frac{(T_4 - T_3) 0.929}{0.688} = 10,920 \text{ lbs. Tension.}$$

$$T_2 = 0.183 \times 20000 \times 2.5 = 9,150 \text{ lbs. Tension.}$$

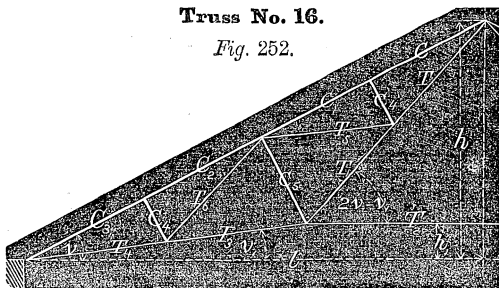
$$T_3 = 10000 \times \frac{50}{20 \times 1} = 25,000 \text{ lbs. Tension.}$$

$$T_4 = 0.683 \times 20000 \times 2.5 = 34,150 \text{ lbs. Tension.}$$

$$T_5 = 0.866 \times 20000 \times 2.5 = 43,300 \text{ lbs. Tension.}$$

Truss No. 16.

Fig. 252.



$$C = C_1 - \frac{2}{7} W \sin. v$$

$$C_1 = C_2 - \frac{18}{70} W \sin. v.$$

$$C_2 = \frac{9}{10} W \frac{\cos. v_1}{\sin. (v - v_1)} - \frac{2}{7} W \sin. v$$

$$C_3 = \frac{9}{10} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$C_4 = \frac{2}{7} W \cos. v.$$

$$C_5 = \frac{16}{70} W \cos. v + \frac{2}{7} W \cos. v = \frac{18}{35} W \cos. v$$

$$T = \left[\frac{9}{10} W \frac{\cos. v_1 \sin. v}{\sin. (v - v_1)} - \frac{4}{5} W \sin.^2 v - \frac{1}{10} W \right] \frac{1}{\sin. (2v - v_1)}$$

$$T_1 = T - \frac{1}{7} W \frac{\cos. v}{\sin. (v - v_1)} = T - T_5$$

$$T_2 = \frac{W}{2} \frac{l}{h - h_1}$$

$$T_3 = \frac{9}{10} W \frac{\cos. v}{\sin. (v - v_1)} - T_5 = \frac{53}{70} W \frac{\cos. v_1}{\sin. (v - v_1)}$$

$$T_4 = \frac{9}{10} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$T_5 = T_6 = T - T_1 = \frac{1}{7} W \frac{\cos. v}{\sin. (v - v_1)}$$

$$T_6 = T_5$$

EXAMPLE.Let $W = 20,000$ lbs. $h = 20$ feet. $h_1 = 0$. $l = 50$ feet. $v = 21^\circ 40'$. $v_1 = 0$. $C = 41885 - 0.286 \times 20000 \times 0.369 = 39,774$ lbs. Compression. $C_1 = 43567 - 0.228 \times 20000 \times 0.369 = 41,885$ lbs. Compression.

$$C_c = 48780 - 5213 = 43,567 \text{ lbs. Compression.}$$

$$C_3 = 0.9 \times 20000 \frac{1}{0.369} = 48,780 \text{ lbs. Compression.}$$

$$C_1 = 0.286 \times 20000 \times 0.929 = 5,213 \text{ lbs. Compression.}$$

$$C_5 = 0.514 \times 20000 \times 0.929 = 9,550 \text{ lbs. Compression.}$$

$$T = \left(0.9 \times 20000 \frac{0.369}{0.369} - 0.8 \times 20000 \times 0.369^2 - 0.1 \times 20000 \right) \frac{1}{0.686} = 20,000 \text{ lbs. Tension.}$$

$$T_1 = T - T_5 = 20000 - 7188 = 12,812 \text{ lbs. Tension.}$$

$$T_2 = \frac{20000}{2} \times \frac{50}{20} = 25,000 \text{ lbs. Tension.}$$

$$T_3 = T_4 - T_5 = 0.757 \times 20000 \frac{0.929}{0.369} = 38,118 \text{ lbs. Tension.}$$

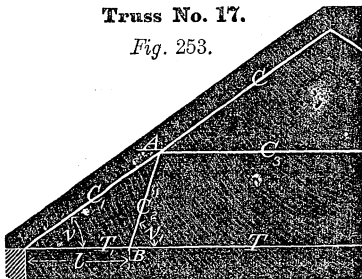
$$T_4 = 0.9 \times 20000 \frac{0.929}{0.369} = 45,306 \text{ lbs. Tension.}$$

$$T_5 = T_6 = T - T_1 = 7.188 \text{ lbs. Tension.}$$

$$T_6 = T_5 = 7,188 \text{ lbs. Tension.}$$

Truss No. 17.

Fig. 253.



When the rafter is resting on joint *A*:

$$C = \frac{W}{4 \sin. v} \qquad C_3 = \frac{1}{2} \frac{W \cos. v \cos. (v_1 - v)}{\sin. v_1}$$

$$C_1 = \frac{W}{4 \sin v} + \frac{1}{2} W \sin v \quad T = C_2 \cos v_1 + T_1$$

$$C_2 = \frac{1}{2} \frac{W \cos. v^2}{\sin. v_1} \quad T_1 = C_1 \cos. v$$

Bending moment at point $B = C_2 \sin. v_1 \cdot l$.

When rafter is fixed at joint *A*:

$$C = \frac{W}{4 \sin. v}$$

$$C_3 = \frac{1}{2} \frac{W \cos. v \cos. (v_1 - v)}{\sin. v_1}$$

$$C_1 = C$$

$$T = \frac{1}{2} W \cotg. v_1 + T_1$$

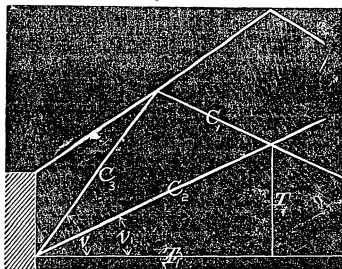
$$C_2 = \frac{1}{2} \frac{W}{\sin. v_1}$$

$$T_1 = \frac{W}{4} \cotg. v$$

$$\text{Bending moment at } B = \frac{W}{2} \cdot l$$

Truss No. 18.

Fig. 254.



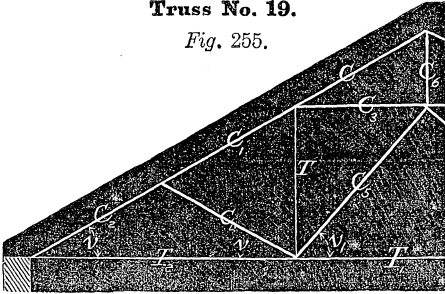
$$C_1 = \frac{1}{2} \frac{W \cos. v_1}{\sin. (v + v_1)}$$

$$C_2 = \frac{1}{2} \frac{W}{\sin. v_1} + C_1$$

$$C_3 = \frac{1}{2} \frac{W \cos. v_2}{\sin. (v + v_1)}$$

$$T = 0$$

$$T_1 = C_3 \cos. v + C_2 \cos. v_1$$

Truss No. 19.*Fig. 255.*

$$C = \frac{1}{2} W \operatorname{cosec}. v$$

$$C_1 = \frac{41}{60} W \operatorname{cosec}. v$$

$$C_2 = \frac{13}{15} W \operatorname{cosec}. v$$

$$C_3 = \frac{2}{3} W \cotg. v$$

$$C_4 = \frac{1}{6} W \cotg. v$$

$$C_5 = \frac{1}{6} W \tan g. v_1$$

$$C_6 = \frac{1}{3} W$$

$$T = \frac{1}{3} W$$

$$T_1 = \frac{2}{3} W \cotg. v + \frac{1}{6} W \tan g. v_1$$

$$T_2 = \frac{5}{6} W \cotg. v$$

EXAMPLE.

Let $W = 20,000$ lbs. $v = 21^\circ 40'$. $v_1 = 56^\circ 30'$.

$$C = 27,000 \text{ lbs. Compression.} \quad C_5 = 3,533 \text{ lbs. Compression.}$$

$$C_1 = 36,900 \text{ lbs. Compression.} \quad C_6 = 6,666 \text{ lbs. Compression.}$$

$$C_2 = 46,800 \text{ lbs. Compression.} \quad T = 6,666 \text{ lbs. Tension.}$$

$$C_3 = 33,466 \text{ lbs. Compression.} \quad T_1 = 37,000 \text{ lbs. Tension.}$$

$$C_4 = 6,867 \text{ lbs. Compression.} \quad T_2 = 41,831 \text{ lbs. Tension.}$$

TABLE OF CONSTANTS.

(For Strains in Roof Trusses of those forms most in use, derived from foregoing formulas.)

Reference.

L = Span in feet. h = Height in feet. C = Compression in member. T = Tension in member.
 W = Weight in lbs. equally distributed over a rafter, to be multiplied by constant for strain in respective member.
 v = Angle between horizontal and rafter.

REFERENCE TO FIGURES.	$\frac{L}{h} = 2$	$\frac{L}{h} = 3$	$\frac{L}{h} = 4$	$\frac{L}{h} = 5$	$\frac{L}{h} = 6$	$\frac{L}{h} = 7$	$\frac{L}{h} = 8$	$\frac{L}{h} = 9$	$\frac{L}{h} = 10$
	$v = 45^\circ$	$v = 33^\circ 40'$	$v = 26^\circ 30'$	$v = 21^\circ 45'$	$v = 18^\circ 20'$	$v = 15^\circ 50'$	$v = 14^\circ 15'$	$v = 12^\circ 30'$	$v = 11^\circ 10'$
Truss No. 1. (See Fig. 237, page 156.)	$C = 1.060$ $T = 0.500$	$C = 1.178$ $T = 0.750$	$C = 1.333$ $T = 1.000$	$C = 1.535$ $T = 1.250$	$C = 1.746$ $T = 1.500$	$C = 1.969$ $T = 1.750$	$C = 2.154$ $T = 2.000$	$C = 2.417$ $T = 2.250$	$C = 2.678$ $T = 2.500$
Truss No. 2. (See Fig. 238, page 157.)	$C = 0.353$ $C_1 = 0.500$ $T = 0.250$	$C = 0.277$ $C_1 = 0.692$ $T = 0.261$	$C = 0.223$ $C_1 = 0.800$ $T = 0.199$	$C = 0.185$ $C_1 = 0.863$ $T = 0.172$	$C = 0.157$ $C_1 = 0.902$ $T = 0.149$	$C = 0.136$ $C_1 = 0.925$ $T = 0.131$	$C = 0.123$ $C_1 = 0.940$ $T = 0.119$	$C = 0.108$ $C_1 = 0.952$ $T = 0.105$	$C = 0.096$ $C_1 = 0.962$ $T = 0.095$
Truss No. 5. (See Fig. 241, page 159.)	$C = 1.456$ $C_1 = 0.500$ $T = 0.750$	$C = 1.465$ $C_1 = 0.750$ $T = 1.125$	$C = 1.820$ $C_1 = 1.000$ $T = 1.500$	$C = 2.194$ $C_1 = 1.250$ $T = 1.875$	$C = 2.576$ $C_1 = 1.500$ $T = 2.250$	$C = 2.974$ $C_1 = 1.750$ $T = 2.625$	$C = 3.315$ $C_1 = 2.000$ $T = 3.000$	$C = 3.754$ $C_1 = 2.250$ $T = 3.375$	$C = 4.193$ $C_1 = 2.500$ $T = 3.750$

$h_1 = \frac{h}{2}$	$C = 2.234$	$C_1 = 2.951$	$C_2 = 3.658$	$C_3 = 4.363$	$C_4 = 5.135$	$C_5 = 5.916$	$C_6 = 6.549$	$C_7 = 7.524$	$C_8 = 8.450$
Truss No. 6. (See Fig. 242, page 159.)	$C_1 = 0.625$	$C_2 = 0.937$	$C_3 = 1.250$	$C_4 = 1.563$	$C_5 = 1.875$	$C_6 = 2.187$	$C_7 = 2.500$	$C_8 = 2.812$	$C_9 = 3.125$
	$T = 1.625$	$T_1 = 2.428$	$T_2 = 3.250$	$T_3 = 4.180$	$T_4 = 5.012$	$T_5 = 5.844$	$T_6 = 6.676$	$T_7 = 7.508$	$T_8 = 8.340$
	$T_1 = 1.625$	$T_2 = 1.625$	$T_3 = 1.625$	$T_4 = 1.625$	$T_5 = 1.625$	$T_6 = 1.625$	$T_7 = 1.625$	$T_8 = 1.625$	$T_9 = 1.625$

$v_1 = v$	$C_1 = 1.145$	$C_2 = 1.462$	$C_3 = 1.820$	$C_4 = 2.194$	$C_5 = 2.575$	$C_6 = 2.974$	$C_7 = 3.313$	$C_8 = 3.754$	$C_9 = 4.192$
Truss No. 7. (See Fig. 243, page 160.)	$C_2 = 0.442$	$C_3 = 0.560$	$C_4 = 0.700$	$C_5 = 0.844$	$C_6 = 0.987$	$C_7 = 1.144$	$C_8 = 1.275$	$C_9 = 1.444$	$C_{10} = 1.612$
	$T = 0.625$	$T_1 = 0.625$	$T_2 = 0.625$	$T_3 = 0.625$	$T_4 = 0.625$	$T_5 = 0.625$	$T_6 = 0.625$	$T_7 = 0.625$	$T_8 = 0.625$
	$T_1 = 0.812$	$T_2 = 1.219$	$T_3 = 1.625$	$T_4 = 2.031$	$T_5 = 2.437$	$T_6 = 2.843$	$T_7 = 3.250$	$T_8 = 3.656$	$T_9 = 4.062$

Truss No. 9. (See Fig. 245, page 162.)	$C_1 = 1.145$	$C_2 = 1.462$	$C_3 = 1.820$	$C_4 = 2.194$	$C_5 = 2.575$	$C_6 = 2.974$	$C_7 = 3.313$	$C_8 = 3.754$	$C_9 = 4.192$
	$C_2 = 0.442$	$C_3 = 0.520$	$C_4 = 0.559$	$C_5 = 0.581$	$C_6 = 0.594$	$C_7 = 0.600$	$C_8 = 0.606$	$C_9 = 0.610$	$C_{10} = 0.613$
	$T = 0.312$	$T_1 = 0.469$	$T_2 = 0.625$	$T_3 = 0.781$	$T_4 = 0.937$	$T_5 = 1.094$	$T_6 = 1.250$	$T_7 = 1.406$	$T_8 = 1.562$
	$T_1 = 0.500$	$T_2 = 0.750$	$T_3 = 1.000$	$T_4 = 1.250$	$T_5 = 1.500$	$T_6 = 1.750$	$T_7 = 2.000$	$T_8 = 2.250$	$T_9 = 2.500$
	$T_2 = 0.812$	$T_3 = 1.219$	$T_4 = 1.625$	$T_5 = 2.031$	$T_6 = 2.445$	$T_7 = 2.860$	$T_8 = 3.277$	$T_9 = 3.694$	$T_{10} = 4.111$

$h_2 = \frac{h}{3}$	$C = 1.932$	$C_1 = 2.333$	$C_2 = 2.963$	$C_3 = 3.553$	$C_4 = 4.170$	$C_5 = 4.780$	$C_6 = 5.339$	$C_7 = 6.030$	$C_8 = 6.723$
Truss No. 12. (See Fig. 248, page 163.)	$C_1 = 0.366$	$C_2 = 0.549$	$C_3 = 0.732$	$C_4 = 0.915$	$C_5 = 1.100$	$C_6 = 1.290$	$C_7 = 1.449$	$C_8 = 1.651$	$C_9 = 1.852$
	$C_2 = 0.258$	$C_3 = 0.329$	$C_4 = 0.410$	$C_5 = 0.494$	$C_6 = 0.590$	$C_7 = 0.670$	$C_8 = 0.747$	$C_9 = 0.845$	$C_{10} = 0.944$
	$T = 1.375$	$T_1 = 2.000$	$T_2 = 2.625$	$T_3 = 3.250$	$T_4 = 3.875$	$T_5 = 4.500$	$T_6 = 5.125$	$T_7 = 5.750$	$T_8 = 6.375$
	$T_1 = 1.233$	$T_2 = 1.233$	$T_3 = 1.233$	$T_4 = 1.233$	$T_5 = 1.233$	$T_6 = 1.233$	$T_7 = 1.233$	$T_8 = 1.233$	$T_9 = 1.233$

TABLE OF CONSTANTS—Continued.

REFERENCE TO FIGURES.	$\frac{L}{h} = 2$	$\frac{L}{h} = 3$	$\frac{L}{h} = 4$	$\frac{L}{h} = 5$	$\frac{L}{h} = 6$	$\frac{L}{h} = 7$	$\frac{L}{h} = 8$	$\frac{L}{h} = 9$	$\frac{L}{h} = 10$
	$v = 45^\circ$	$v = 33^\circ 40'$	$v = 26^\circ 30'$	$v = 21^\circ 45'$	$v = 18^\circ 20'$	$v = 15^\circ 50'$	$v = 14^\circ 15'$	$v = 12^\circ 30'$	$v = 11^\circ 10'$
$v_1 = 0$									
Truss No. 13. (See Fig. 249, page 166.)	$C_2 = 1.211$ $C_3 = 0.404$ $C_4 = 0.261$ $T_1 = 0.683$ $T_2 = 0.866$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 1.559$ $C_3 = 0.460$ $C_4 = 0.330$ $T_1 = 1.024$ $T_2 = 1.299$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 1.940$ $C_3 = 0.523$ $C_4 = 0.413$ $T_1 = 1.366$ $T_2 = 1.732$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 2.338$ $C_3 = 0.594$ $C_4 = 0.495$ $T_1 = 1.707$ $T_2 = 2.165$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 2.745$ $C_3 = 0.660$ $C_4 = 0.567$ $T_1 = 1.956$ $T_2 = 2.606$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 3.170$ $C_3 = 0.731$ $C_4 = 0.661$ $T_1 = 2.304$ $T_2 = 3.048$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 3.533$ $C_3 = 0.797$ $C_4 = 0.742$ $T_1 = 2.705$ $T_2 = 3.429$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 4.000$ $C_3 = 0.902$ $C_4 = 0.837$ $T_1 = 3.080$ $T_2 = 3.905$ $T_3 = 0.734$ $T_5 = 0.183$	$C_2 = 4.468$ $C_3 = 0.990$ $C_4 = 0.936$ $T_1 = 3.456$ $T_2 = 4.382$ $T_3 = 0.734$ $T_5 = 0.183$
$H = 0$ $h = D$	$C_4 = 1.269$ $C_5 = 0.420$ $C_6 = 0.285$ $C_7 = 0.203$ $T_1 = 0.906$ $T_2 = 0.757$ $T_3 = 0.631$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 1.620$ $C_5 = 0.476$ $C_6 = 0.305$ $C_7 = 0.257$ $T_1 = 1.350$ $T_2 = 1.136$ $T_3 = 0.921$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 2.016$ $C_5 = 0.447$ $C_6 = 0.349$ $C_7 = 0.320$ $T_1 = 1.806$ $T_2 = 1.514$ $T_3 = 1.267$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 2.430$ $C_5 = 0.521$ $C_6 = 0.393$ $C_7 = 0.386$ $T_1 = 2.250$ $T_2 = 1.893$ $T_3 = 1.589$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 2.853$ $C_5 = 0.563$ $C_6 = 0.428$ $C_7 = 0.451$ $T_1 = 2.710$ $T_2 = 2.279$ $T_3 = 1.926$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 3.294$ $C_5 = 0.616$ $C_6 = 0.515$ $C_7 = 0.522$ $T_1 = 3.168$ $T_2 = 2.665$ $T_3 = 2.222$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 3.672$ $C_5 = 0.670$ $C_6 = 0.571$ $C_7 = 0.582$ $T_1 = 3.560$ $T_2 = 2.998$ $T_3 = 2.496$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 4.158$ $C_5 = 0.731$ $C_6 = 0.605$ $C_7 = 0.660$ $T_1 = 4.059$ $T_2 = 3.415$ $T_3 = 2.862$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$	$C_4 = 4.644$ $C_5 = 0.781$ $C_6 = 0.667$ $C_7 = 0.737$ $T_1 = 4.554$ $T_2 = 3.831$ $T_3 = 3.214$ $T_4 = 0.800$ $T_5 = 0.253$ $T_6 = 0.143$
Truss No. 14. (See Fig. 250, page 167.)									

$v_1 = 0$ $T = 0$ $h_1 = 0$	$C_2 = 1.560$ $C_3 = 1.940$ $C_2 = 2.340$ $C_2 = 2.745$ $C_2 = 3.170$ $C_2 = 3.533$ $C_2 = 4.000$ $C_2 = 4.170$ $C_3 = 0.457$ $C_3 = 0.404$ $C_3 = 0.511$ $C_3 = 0.523$ $C_3 = 0.528$ $C_3 = 0.534$ $C_3 = 0.537$ $C_3 = 0.539$ $C_4 = 0.305$ $C_4 = 0.328$ $C_4 = 0.341$ $C_4 = 0.349$ $C_4 = 0.352$ $C_4 = 0.356$ $C_4 = 0.359$ $C_4 = 0.360$ $T_1 = 0.376$ $T_1 = 0.446$ $T_1 = 0.546$ $T_1 = 0.627$ $T_1 = 0.711$ $T_1 = 0.788$ $T_1 = 0.885$ $T_1 = 1.015$ $T_2 = 0.274$ $T_2 = 0.366$ $T_2 = 0.457$ $T_2 = 0.551$ $T_2 = 0.644$ $T_2 = 0.725$ $T_2 = 0.825$ $T_2 = 0.926$ $T_3 = 0.750$ $T_3 = 1.000$ $T_3 = 1.250$ $T_3 = 1.500$ $T_3 = 1.750$ $T_3 = 2.000$ $T_3 = 2.250$ $T_3 = 2.500$ $T_4 = 1.025$ $T_4 = 1.366$ $T_4 = 1.710$ $T_4 = 2.056$ $T_4 = 2.404$ $T_4 = 2.705$ $T_4 = 3.080$ $T_4 = 3.456$ $T_5 = 1.300$ $T_5 = 1.732$ $T_5 = 2.165$ $T_5 = 2.607$ $T_5 = 3.048$ $T_5 = 3.430$ $T_5 = 3.906$ $T_5 = 4.382$									
	Truss No. 15. (See Fig. 251, page 168.)									
$v_1 = 0$ $h_1 = 0$	$C_3 = 1.269$ $C_3 = 2.620$ $C_3 = 2.016$ $C_3 = 3.853$ $C_3 = 3.294$ $C_3 = 3.672$ $C_3 = 4.158$ $C_3 = 4.644$ $C_4 = 0.202$ $C_4 = 0.238$ $C_4 = 0.255$ $C_4 = 0.271$ $C_4 = 0.274$ $C_4 = 0.277$ $C_4 = 0.279$ $C_4 = 0.280$ $C_5 = 0.363$ $C_5 = 0.428$ $C_5 = 0.460$ $C_5 = 0.488$ $C_5 = 0.493$ $C_5 = 0.499$ $C_5 = 0.502$ $C_5 = 0.504$ $T_1 = 0.400$ $T_1 = 0.600$ $T_1 = 0.800$ $T_1 = 1.000$ $T_1 = 1.400$ $T_1 = 1.600$ $T_1 = 1.800$ $T_1 = 2.000$ $T_2 = 0.257$ $T_2 = 0.386$ $T_2 = 0.514$ $T_2 = 0.643$ $T_2 = 0.770$ $T_2 = 1.034$ $T_2 = 1.155$ $T_2 = 1.276$ $T_3 = 0.500$ $T_3 = 0.750$ $T_3 = 1.000$ $T_3 = 1.250$ $T_3 = 1.500$ $T_3 = 2.000$ $T_3 = 2.250$ $T_3 = 2.500$ $T_4 = 0.700$ $T_4 = 1.050$ $T_4 = 1.400$ $T_4 = 1.750$ $T_4 = 2.100$ $T_4 = 2.450$ $T_4 = 2.800$ $T_4 = 3.150$ $T_5 = 0.900$ $T_5 = 1.350$ $T_5 = 1.800$ $T_5 = 2.250$ $T_5 = 2.700$ $T_5 = 3.168$ $T_5 = 3.564$ $T_5 = 4.059$ $T_6 = 0.143$ $T_6 = 0.214$ $T_6 = 0.286$ $T_6 = 0.357$ $T_6 = 0.430$ $T_6 = 0.503$ $T_6 = 0.567$ $T_6 = 0.645$ $T_7 = 0.143$ $T_7 = 0.214$ $T_7 = 0.286$ $T_7 = 0.357$ $T_7 = 0.430$ $T_7 = 0.503$ $T_7 = 0.567$ $T_7 = 0.645$									
	Truss No. 16. (See Fig. 252, page 170.)									
	$C_3 = 1.269$ $C_3 = 2.620$ $C_3 = 2.016$ $C_3 = 3.853$ $C_3 = 3.294$ $C_3 = 3.672$ $C_3 = 4.158$ $C_3 = 4.644$ $C_4 = 0.202$ $C_4 = 0.238$ $C_4 = 0.255$ $C_4 = 0.271$ $C_4 = 0.274$ $C_4 = 0.277$ $C_4 = 0.279$ $C_4 = 0.280$ $C_5 = 0.363$ $C_5 = 0.428$ $C_5 = 0.460$ $C_5 = 0.488$ $C_5 = 0.493$ $C_5 = 0.499$ $C_5 = 0.502$ $C_5 = 0.504$ $T_1 = 0.400$ $T_1 = 0.600$ $T_1 = 0.800$ $T_1 = 1.000$ $T_1 = 1.400$ $T_1 = 1.600$ $T_1 = 1.800$ $T_1 = 2.000$ $T_2 = 0.257$ $T_2 = 0.386$ $T_2 = 0.514$ $T_2 = 0.643$ $T_2 = 0.770$ $T_2 = 1.034$ $T_2 = 1.155$ $T_2 = 1.276$ $T_3 = 0.500$ $T_3 = 0.750$ $T_3 = 1.000$ $T_3 = 1.250$ $T_3 = 1.500$ $T_3 = 2.000$ $T_3 = 2.250$ $T_3 = 2.500$ $T_4 = 0.700$ $T_4 = 1.050$ $T_4 = 1.400$ $T_4 = 1.750$ $T_4 = 2.100$ $T_4 = 2.450$ $T_4 = 2.800$ $T_4 = 3.150$ $T_5 = 0.900$ $T_5 = 1.350$ $T_5 = 1.800$ $T_5 = 2.250$ $T_5 = 2.700$ $T_5 = 3.168$ $T_5 = 3.564$ $T_5 = 4.059$ $T_6 = 0.143$ $T_6 = 0.214$ $T_6 = 0.286$ $T_6 = 0.357$ $T_6 = 0.430$ $T_6 = 0.503$ $T_6 = 0.567$ $T_6 = 0.645$ $T_7 = 0.143$ $T_7 = 0.214$ $T_7 = 0.286$ $T_7 = 0.357$ $T_7 = 0.430$ $T_7 = 0.503$ $T_7 = 0.567$ $T_7 = 0.645$									

EXAMPLE TO TABLE OF CONSTANTS. (*Truss No. 13.*)

What is the amount of strain in the various members of a truss, according to *Fig. 249*, of the following dimensions, viz: Span 60 feet, distance between trusses 10 feet, height at center 10 feet, weight to be carried, including weight of construction, $66\frac{2}{3}$ lbs. per square foot horizontally; hence total weight on one rafter $= 30 \times 10 \times 66\frac{2}{3} = 20,000$ lbs.?

$$\begin{array}{lll} L = 60 \text{ feet.} & \frac{L}{h} = \frac{60}{10} = 6. & v = 18^\circ 20'. \\ h = 10 \text{ feet.} & & W = 20,000 \text{ lbs.} \end{array}$$

Member.	Constant.	W	Strains.	
C_2	$= 2.745 \times$	$20,000$	$= 54,900$ lbs.	} Compression.
C_3	$= 0.660 \times$	$20,000$	$= 13,200$ lbs.	
C_4	$= 0.567 \times$	$20,000$	$= 11,340$ lbs.	
T	$= 1.956 \times$	$20,000$	$= 39,120$ lbs.	} Tension.
T_1	$= 2.606 \times$	$20,000$	$= 52,120$ lbs.	
T_2	$= 0.734 \times$	$20,000$	$= 14,680$ lbs.	
T_3	$= 0.183 \times$	$20,000$	$= 3,660$ lbs.	

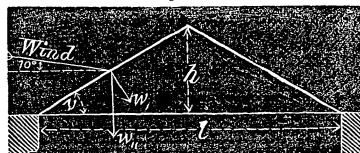
[NOTE.—In the foregoing table the proportion of h to L is approximate. The constants are based on the angles.]

PRESSURE OF WIND ON ROOFS.

In the following table the maximum pressure of wind is taken at 50 lbs. per square foot:

The angle between horizontal and direction of wind is generally $10^\circ 00'$. (See diagram.)

Fig. 256.



Reference.

F = Force of wind in lbs. = 50.

w = Pressure at right angles to surface per square foot in lbs.

w_v = Pressure, vertical, per square foot in lbs.

$$w_v = F \sin^2(v + 10)$$

$$w_v = \frac{w}{\cos. v}$$

Proportion of height h to span l .	Angle v .	Pressure w_1 in lbs.	Pressure w_{II} in lbs.
$h = \frac{l}{0}$	90° 00'	50.00	0.00
$h = \frac{l}{2}$	45° 00'	33.53	47.40
$h = \frac{l}{3}$	33° 41' 50''	23.80	28.60
$h = \frac{l}{4}$	26° 33' 50''	17.64	19.70
$h = \frac{l}{5}$	21° 48'	13.83	14.80
$h = \frac{l}{6}$	18° 26'	11.23	11.80
$h = \frac{l}{7}$	15° 54' 40''	9.46	9.80
$h = \frac{l}{8}$	14° 02' 10''	8.56	8.80
$h = \frac{l}{9}$	12° 31' 40''	7.29	7.40
$h = \frac{l}{10}$	11° 18' 40''	6.51	6.60

PRESSURE OF SNOW ON ROOFS.

The average pressure of snow on a level surface, in the United States, is about 15 lbs. per square foot.

The following table gives the pressure per square foot on various inclined surfaces:

Reference.

P = Pressure per square foot in lbs. = 15.

p_1 = Vertical pressure in lbs.

p_2 = Pressure at right angles to surface in lbs.

v = Angle between surface and horizontal.

$p_1 = P \cos. v.$

$p_2 = P_1 \cos. v.$

Proportion of height h to span l .	Angle v .	Pressure P_1 in lbs.	Pressure P_2 in lbs.
$h = \frac{l}{2}$	45° 00'	10.60	7.49
$h = \frac{l}{3}$	33° 41' 50''	12.48	10.38
$h = \frac{l}{4}$	26° 33' 50''	13.42	12.00
$h = \frac{l}{5}$	21° 48'	13.93	12.94
$h = \frac{l}{6}$	18° 26'	14.23	13.50
$h = \frac{l}{7}$	15° 54' 40''	14.41	13.86
$h = \frac{l}{8}$	14° 02' 10''	14.52	14.05
$h = \frac{l}{9}$	12° 31' 40''	14.64	14.29
$h = \frac{l}{10}$	11° 18' 40''	14.71	14.43
$h = \frac{l}{\infty}$	0° 00' 00''	15.00	15.00

TIE RODS AND BARS.

Capacity and Proportional Dimensions of Wrought-iron Tie Rods, Tie Bars, and Pins or Bolts.

Ultimate resistance to tearing = 60,000 lbs. = 30 tons per square inch.

Ultimate resistance to shearing = 50,000 lbs. = 25 tons per square inch. (See *Fig.* 258.)

Capacity of tie or bar.				Sectional area in sq. inches.	Diameter in inches, if round.	Dimension of flat bars in in., uniform thickness.			Diameter <i>D</i> of pin or bolt.	
3 times safety.		5 times safety.				Thickness <i>t</i> of bar.	Width <i>b</i> of bar.	Width <i>b</i> around eye.	One place of shearing.	Two places of shearing.
Lbs.	Tons.	Lbs.	Tons.							
5,000	2.50	3,000	1.50	0.25	0.56	$\frac{1}{4}$	1	0.75	0.62	0.44
6,200	3.10	3,720	1.86	0.31	0.62	"	$1\frac{1}{4}$	0.93	0.69	0.48
7,400	3.70	4,440	2.22	0.37	0.70	"	$1\frac{1}{2}$	1.12	0.75	0.53
8,600	4.30	5,160	2.58	0.43	0.74	"	$1\frac{3}{4}$	1.31	0.80	0.57
10,000	5.00	6,000	3.00	0.50	0.79	"	2	1.50	0.88	0.62
11,200	5.60	6,720	3.36	0.56	0.84	"	$2\frac{1}{4}$	1.68	0.92	0.65
12,400	6.20	7,440	3.72	0.62	0.89	"	$2\frac{1}{2}$	1.87	0.97	0.69
13,600	6.80	8,160	3.88	0.68	0.93	"	$2\frac{3}{4}$	2.06	1.01	0.72
15,000	7.50	9,000	4.50	0.75	0.97	"	3	2.25	1.08	0.76
7,400	3.70	4,440	2.22	0.37	0.68	$\frac{3}{8}$	1	0.75	0.75	0.53
9,200	4.60	5,520	2.76	0.46	0.76	"	$1\frac{1}{4}$	0.93	0.83	0.58
11,200	5.60	6,720	3.36	0.56	0.84	"	$1\frac{1}{2}$	1.12	0.92	0.65
13,000	6.50	7,800	3.90	0.65	0.91	"	$1\frac{3}{4}$	1.31	0.99	0.70
15,000	7.50	9,000	4.50	0.75	0.97	"	2	1.50	1.08	0.76
16,800	8.40	10,080	5.04	0.84	1.04	"	$2\frac{1}{4}$	1.68	1.13	0.80
18,600	9.30	11,160	5.58	0.93	1.09	"	$2\frac{1}{2}$	1.87	1.19	0.83
20,600	10.30	12,360	6.18	1.03	1.15	"	$2\frac{3}{4}$	2.06	1.24	0.83
22,400	11.20	13,440	6.72	1.12	1.19	"	3	2.25	1.29	0.92
10,000	5.00	6,000	3.00	0.50	0.79	$\frac{1}{2}$	1	0.75	0.88	0.62
12,400	6.20	7,440	3.72	0.62	0.88	"	$1\frac{1}{4}$	0.93	0.97	0.69
15,000	7.50	9,000	4.50	0.75	0.97	"	$1\frac{1}{2}$	1.12	1.08	0.76
17,400	8.70	10,440	5.02	0.87	1.05	"	$1\frac{3}{4}$	1.31	1.16	0.82
20,000	10.00	12,000	6.00	1.00	1.13	"	2	1.50	1.24	0.88
22,400	11.20	13,440	6.72	1.12	1.20	"	$2\frac{1}{4}$	1.68	1.32	0.93
25,000	12.50	15,000	7.50	1.25	1.26	"	$2\frac{1}{2}$	1.87	1.39	0.98
27,400	13.70	16,440	8.22	1.37	1.32	"	$2\frac{3}{4}$	2.06	1.45	1.03
30,000	15.00	18,000	9.00	1.50	1.39	"	3	2.25	1.52	1.08
12,400	6.20	7,440	3.72	0.62	0.90	$\frac{5}{8}$	1	0.75	0.98	0.69
15,600	7.80	9,360	4.68	0.78	1.00	"	$1\frac{1}{4}$	0.93	1.09	0.77
18,600	9.30	11,160	5.58	0.93	1.09	"	$1\frac{1}{2}$	1.12	1.20	0.85
21,800	10.90	13,080	6.54	1.09	1.18	"	$1\frac{3}{4}$	1.31	1.29	0.91
25,000	12.50	15,000	7.50	1.25	1.26	"	2	1.50	1.39	0.98
28,000	14.00	16,800	8.40	1.40	1.34	"	$2\frac{1}{4}$	1.68	1.47	1.04
30,533	15.27	18,720	9.36	1.56	1.41	"	$2\frac{1}{2}$	1.87	1.54	1.09

Capacity of tie or bar.				Sectional area in sq. inches.	Diameter in inches, if round.	Dimension of flat bars in in., uniform thickness.			Diameter <i>D</i> of pin or bolt.	
3 times safety.		5 times safety.				Thickness <i>t</i> of bar	Width <i>b</i> ₁ of bar.	Width <i>b</i> around eye.	One place of shearing.	Two places of shearing.
Lbs.	Tons.	Lbs.	Tons.							
34,200	17.10	20,520	10.26	1.71	1.48	$\frac{5}{8}$	$2\frac{3}{4}$	2.06	1.62	1.14
37,500	18.75	22,440	11.22	1.87	1.54		3	2.25	1.69	1.20
15,000	7.50	9,000	4.50	0.75	0.98	$\frac{3}{4}$	1	0.75	1.08	0.76
18,600	9.30	11,160	5.58	0.93	1.09		$1\frac{1}{4}$	0.93	1.20	0.85
22,400	11.20	13,440	6.72	1.12	1.19	"	$1\frac{1}{2}$	1.12	1.31	0.93
26,200	13.10	15,720	7.86	1.31	1.30	"	$1\frac{3}{4}$	1.31	1.41	1.00
30,000	15.00	18,000	9.00	1.50	1.39	"	2	1.50	1.52	1.08
33,600	16.80	20,160	10.08	1.68	1.46	"	$2\frac{1}{4}$	1.68	1.62	1.14
37,400	18.70	22,440	11.22	1.87	1.54	"	$2\frac{1}{2}$	1.87	1.69	1.20
41,200	20.60	24,720	12.36	2.06	1.62	"	$2\frac{3}{4}$	2.06	1.77	1.26
45,000	22.50	27,000	13.50	2.25	1.69	"	3	2.25	1.86	1.32
17,400	8.70	10,440	5.22	0.87	1.05	$\frac{7}{8}$	1	0.75	1.16	0.82
21,800	10.90	13,080	6.54	1.09	1.18	"	$1\frac{1}{4}$	0.93	1.29	0.91
26,200	13.10	15,720	7.86	1.31	1.29	"	$1\frac{1}{2}$	1.12	1.41	1.00
30,600	15.30	18,360	9.18	1.53	1.40	"	$1\frac{3}{4}$	1.31	1.53	1.08
34,800	17.40	20,880	10.44	1.74	1.49	"	2	1.50	1.63	1.16
39,200	19.60	23,520	11.76	1.96	1.58	"	$2\frac{1}{4}$	1.68	1.73	1.23
43,600	21.80	26,160	13.08	2.18	1.66	"	$2\frac{1}{2}$	1.87	1.82	1.29
48,000	24.00	28,800	14.40	2.40	1.75	"	$2\frac{3}{4}$	2.06	1.89	1.34
52,400	26.20	31,440	15.72	2.62	1.83	"	3	2.25	2.00	1.42
20,000	10.00	12,000	6.00	1.00	1.13	1	1	0.75	1.39	0.80
25,000	12.50	15,000	7.50	1.25	1.26	"	$1\frac{1}{4}$	0.93	1.45	0.98
30,000	15.00	18,000	9.00	1.50	1.39	"	$1\frac{1}{2}$	1.12	1.52	1.08
35,000	17.50	21,000	10.50	1.75	1.49	"	$1\frac{3}{4}$	1.31	1.64	1.16
40,000	20.00	24,000	12.00	2.00	1.60	"	2	1.50	1.75	1.24
45,000	22.50	27,000	13.50	2.25	1.70	"	$2\frac{1}{4}$	1.68	1.86	1.32
50,000	25.00	30,000	15.00	2.50	1.79	"	$2\frac{1}{2}$	1.87	1.96	1.39
55,000	27.50	33,000	16.50	2.75	1.87	"	$2\frac{3}{4}$	2.06	2.05	1.45
60,000	30.00	36,000	18.00	3.00	1.96	"	3	2.25	2.15	1.52
28,000	14.00	16,800	8.40	1.40	1.34	$1\frac{1}{8}$	$1\frac{1}{4}$	0.93	1.47	1.04
33,600	16.80	20,160	10.08	1.68	1.47	"	$1\frac{1}{2}$	1.12	1.60	1.13
39,600	19.80	23,520	11.76	1.98	1.58	"	$1\frac{3}{4}$	1.31	1.73	1.23
45,000	22.50	27,000	13.50	2.25	1.69	"	2	1.50	1.86	1.32
50,600	25.30	30,360	15.18	2.53	1.80	"	$2\frac{1}{4}$	1.68	1.97	1.39
56,200	28.10	33,720	16.86	2.81	1.89	"	$2\frac{1}{2}$	1.87	2.09	1.48
61,800	30.90	37,080	18.54	3.09	1.98	"	$2\frac{3}{4}$	2.06	2.18	1.54
67,400	33.70	40,440	20.22	3.37	2.08	"	3	2.25	2.26	1.60
73,000	36.50	43,800	21.90	3.65	2.16	"	$3\frac{1}{4}$	2.43	2.36	1.67
78,600	39.30	47,160	23.58	3.93	2.24	"	$3\frac{1}{2}$	2.62	2.45	1.74
84,200	42.10	50,520	25.26	4.21	2.32	"	$3\frac{3}{4}$	2.81	2.53	1.80
90,000	45.00	54,000	27.00	4.50	2.40	"	4	3.00	2.63	1.86
31,200	15.60	18,720	9.36	1.56	1.41	$1\frac{1}{4}$	$1\frac{1}{4}$	0.93	1.54	1.09
37,400	18.70	22,440	11.22	1.87	1.55	"	$1\frac{1}{2}$	1.12	1.69	1.20
43,600	21.80	26,160	13.08	2.18	1.67	"	$1\frac{3}{4}$	1.31	1.82	1.29
50,000	25.00	30,000	15.00	2.50	1.79	"	2	1.50	1.96	1.39
56,200	28.10	33,720	16.86	2.81	1.89	"	$2\frac{1}{4}$	1.68	2.09	1.48
62,400	31.20	37,440	18.72	3.12	1.99	"	$2\frac{1}{2}$	1.87	2.19	1.55

Capacity of tie or bar.				Sectional area in sq. inches.	Diameter in inches, if round.	Dimension of flat bars in in., uniform thickness.			Diameter <i>D</i> of pin or bolt.	
3 times safety.		5 times safety.				Thickness <i>t</i> of bar.	Width <i>b</i> ₁ of bar.	Width <i>b</i> around eye.	One place of shearing.	Two places of shearing.
Lbs.	Tons.	Lbs.	Tons.							
68,600	34.30	41,160	20.58	3.43	2.10	1 $\frac{1}{4}$	2 $\frac{3}{4}$	2.06	2.29	1.62
75,000	37.50	45,000	22.50	3.75	2.19	"	3	2.25	2.40	1.70
81,200	40.60	48,720	24.36	4.06	2.27	"	3 $\frac{1}{4}$	2.43	2.49	1.76
87,400	43.70	52,440	26.22	4.37	2.36	"	3 $\frac{1}{2}$	2.62	2.60	1.84
93,600	46.80	56,160	28.08	4.68	2.44	"	3 $\frac{3}{4}$	2.81	2.68	1.89
100,000	50.00	60,000	30.00	5.00	2.53	"	4	3.00	2.77	1.96
41,200	20.60	24,720	12.36	2.06	1.62	1 $\frac{3}{8}$	1 $\frac{1}{2}$	1.12	1.77	1.26
48,000	24.00	28,800	14.40	2.40	1.75	"	1 $\frac{3}{4}$	1.31	1.89	1.34
55,000	27.50	33,000	16.50	2.75	1.87	"	2	1.50	2.05	1.45
61,800	30.90	37,080	18.54	3.09	1.98	"	2 $\frac{1}{4}$	1.68	2.18	1.54
68,600	34.30	41,160	20.58	3.43	2.09	"	2 $\frac{1}{2}$	1.87	2.29	1.62
75,600	37.80	45,360	22.68	3.78	2.19	"	2 $\frac{3}{4}$	2.06	2.41	1.71
82,400	41.20	49,440	24.72	4.12	2.29	"	3	2.25	2.51	1.78
89,200	44.60	53,520	26.76	4.46	2.38	"	3 $\frac{1}{4}$	2.43	2.61	1.85
96,200	48.10	57,720	28.86	4.81	2.47	"	3 $\frac{1}{2}$	2.62	2.71	1.92
103,000	51.50	61,800	30.90	5.15	2.56	"	3 $\frac{3}{4}$	2.81	2.81	1.99
110,000	55.00	66,000	33.00	5.50	2.65	"	4	3.00	2.90	2.05
45,000	22.5	27,000	13.50	2.25	1.70	1 $\frac{1}{2}$	1 $\frac{1}{2}$	1.12	1.86	1.32
52,400	26.20	31,440	15.72	2.62	1.83	"	1 $\frac{3}{4}$	1.31	2.00	1.42
60,000	30.00	36,000	18.00	3.00	1.96	"	2	1.50	2.15	1.52
67,400	33.70	40,440	20.22	3.37	2.07	"	2 $\frac{1}{4}$	1.68	2.27	1.61
75,000	37.50	45,000	22.50	3.75	2.19	"	2 $\frac{1}{2}$	1.87	2.40	1.70
82,400	41.20	49,440	24.72	4.12	2.29	"	2 $\frac{3}{4}$	2.06	2.51	1.78
90,000	45.00	54,000	27.00	4.50	2.40	"	3	2.25	2.63	1.86
97,400	48.70	58,440	29.22	4.87	2.49	"	3 $\frac{1}{4}$	2.43	2.73	1.93
105,000	52.50	63,000	31.50	5.25	2.59	"	3 $\frac{1}{2}$	2.62	2.84	2.01
113,400	56.20	67,440	33.72	5.62	2.67	"	3 $\frac{3}{4}$	2.81	2.93	2.08
120,000	60.00	72,000	36.00	6.00	2.77	"	4	3.00	3.03	2.15
127,400	63.70	76,440	38.22	6.37	2.85	"	4 $\frac{1}{4}$	3.18	3.12	2.21
135,000	67.50	81,000	40.50	6.75	2.93	"	4 $\frac{1}{2}$	3.37	3.22	2.28
142,400	71.20	85,440	42.72	7.12	3.01	"	4 $\frac{3}{4}$	3.55	3.30	2.34
150,000	75.00	90,000	45.00	7.50	3.10	"	5	3.75	3.39	2.40

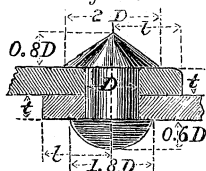
JOINTS OR CONNECTIONS IN IRON CONSTRUCTION.

PROPORTIONS OF BOLTS, NUTS, RIVETS, &c.

Reference.

- A = Sectional area of bolt, rivet, or pin.
 A_1 = Sectional area of all rivets in a joint.
 A_2 = Sectional area of one plate.
 D = Diameter of bolt, rivet, or pin.
 S = Ultimate resistance to shearing of material.
 T = Ultimate resistance to tearing of material.
 T_1 = Tensional strain on joint, &c.
 a = Number of times that a bolt, &c., will have to be sheared.
 (See 2 on *Fig. 258*.)
 d = Distance between centres of rivets.
 k = Factor of safety.
 l = Overlap, approximately $1\frac{2}{3} d$ to $1\frac{3}{4} d$.
 n = Number of rivets in a joint.
 n = Number of lines of rivets in a joint at right angles to strain.
 t = Thickness of a plate.

RIVETS.

Fig. 257.

For tension in direction of rivet:

$$D = \sqrt{\frac{T_1 k}{T \cdot 0.7854}}$$

For shearing at right angles:

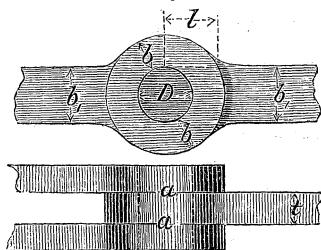
$$\text{If at one place } D = \sqrt{\frac{T_1 k}{S \cdot 0.7854}}$$

$$\text{If at two places } D = \sqrt{\frac{T_1 k}{S \cdot 1.5708}}$$

$$\text{Approximately: } l = 3t \quad D = 3t$$

PIN, &C., IN TIE BARS.

Fig. 258.



$$A = \frac{T_1 k}{S a}$$

$$D = 1.128 \sqrt{A}$$

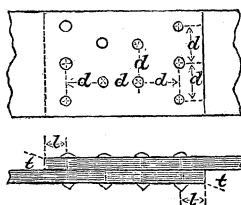
$$b + b = 1.5 b_1$$

$$l = \frac{T_1 k}{2 S t}$$

PLATE JOINTS.

No. 1.—Plate Joint Overlapped, four lines of Rivets.

Fig. 259.



$$d = D + \frac{1}{t} (0.7854 D^2 n)$$

Approximately $d = 1.5t$ to $2t$

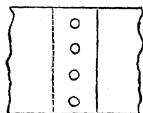
$$A_1 = A_2$$

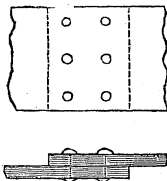
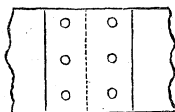
$$D = \frac{1}{m} \sqrt{\frac{T_1 k}{S 0.7854}}$$

$$l = \frac{T_1 k}{2 m t S}$$

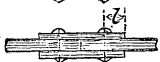
No. 2.—Plate Joint Overlapped, single line of Rivet.

Fig. 260. (Same as No. 1.)



No. 3.—*Plate Joint Overlapped, two lines of Rivets.**Fig. 261.* (Same as No. 1.)**No. 4.**—*Fish Joints, two lines of Rivets.**Fig. 262.*

One fish plate. (Same as No. 1.)



Two fish plates.

Thickness of each fish plate = $\frac{1}{2} t$.

$$D = \frac{1}{m} \sqrt{\frac{T_1 k}{S 1.5708}}$$

(Otherwise same as No. 1.)

DIMENSIONS OF BOLTS AND NUTS.

(Whitworth's proportions.)

Figs. 263, 264, 265, 266, 267, 268, 269, 270, and 271.










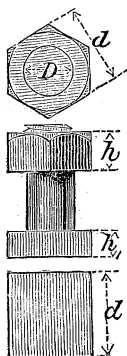
Dia. of Bolt.	Dimension of Nuts and Heads.					Dia. of Core.	No. Threads per inch.	
	Inch.	Inch.	Inch.	Inch.	Inch.			
								
Inch.	Inch.	Inch.	Inch.	Inch.	Inch.	Inch.		
3	4½	5.18	5	7.07	3	2.57	3.5	1.50
2¾	4⅛	4.76	4½	6.37	2¾	2.35	3.5	1.75
2½	3¾	4.33	4⅛	5.83	2½	2.13	4.0	2.00
2¼	3⅝	3.89	3¾	5.30	2¼	1.91	4.0	2.12
2	3	3.46	3⅜	4.76	2	1.69	4.5	2.25
1⅞	2¾	3.17	3	4.24	1⅞	1.58	4.5	2.37
1¾	2⅝	3.03	2¾	3.89	1¾	1.47	5.0	2.50
1⅝	2½	2.98	2⅝	3.71	1⅝	1.36	5.0	2.75
1½	2¼	2.59	2½	3.53	1½	1.25	6.0	3.00
1⅜	2	2.30	2¼	3.18	1⅜	1.14	6.0	3.25
1¼	1⅞	2.16	2	2.82	1¼	1.08	7.0	3.50
1⅓	1⅝	1.87	1⅞	2.64	1⅓	0.92	7.0	4.00
1	1½	1.73	1⅝	2.29	1	0.81	8.0	5.00
⅞	1⅕	1.51	1½	2.12	⅞	0.70	9.0	6.00
¾	1⅓	1.38	1⅕	1.86	¾	0.59	10.0	6.00
⅝	1	1.15	1⅓	1.67	⅝	0.48	11.0	7.00
⅙	⅞	1.01	1	1.41	⅙	0.42	11.0	7.00
½	¾	0.86	⅞	1.23	½	0.37	12.0	8.00
⅗	¾	0.86	¾	1.06	⅗	0.31	14.0	8.00
⅜	⅙	0.64	¾	1.06	⅜	0.26	16.0	9.00
⅙	⅗	0.50	⅙	0.79	⅙	0.20	18.0	9.00
¼	⅜	0.43	⅙	0.79	¼	0.15	20.0	10.00

Fig. 272.



Approximate proportions of bolts, nuts, and heads in inches:

$$d = 1.4 D + 0.25 = \text{Inscribed circle.}$$

$$h = D = \text{Height of nut.}$$

$$h_1 = 0.7 D = \text{Height of head.}$$

COMPOUND STRAINS IN HORIZONTAL AND SLOPING BEAMS.

(Load equally distributed or between supports.)

Area of Cross-section necessary to resist a Cross-breaking and Compressive Strain in Beams acting as a Boom in Trusses, &c., or Beams acting as Rafters, &c.

Reference.

m = Bending moment (See Page 100.)

C = Compressive strain. (See Roof and Simple Trusses.)

q = A factor depending on form of cross-section.

I = Moment of inertia of cross-section.

s = Distance from neutral axis to most compressed fibres.

A = Sectional area of beam, &c.

h = Depth of beam, &c.

p = Resistance to compression with safety per square inch of section.

W = Total load.

l = Length of beam, &c.

$$q = \frac{I}{\frac{s}{h} h^2 A}$$

For horizontal beams, &c.:

$$A = \frac{1}{p} \left(\frac{M}{qh} + C \right) \quad p = \frac{1}{A} \left(\frac{M}{qh} + C \right)$$

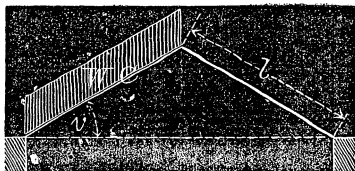
For sloping beams, &c., v = angle between horizontal and beam:

$$A = \frac{W}{p} \left[\frac{1}{2} \left(\frac{1}{\sin. v} + \sin. v \right) + \frac{l \cos. v}{12 qh} \right]$$

$$p = \frac{W}{A} \left[\frac{1}{2} \left(\frac{1}{\sin. v} + \sin. v \right) + \frac{l \cos. v}{12 qh} \right]$$

RAFTER OF A ROOF TRUSS.

Fig. 273.



EXAMPLE.

Reference.

$W = 2.5$ tons. $C = 2.8$ tons. $l = 10$ feet. $v = 26^\circ 30'$
 $p = 5$ tons per square inch.

We will assume a Phoenix Co's six-inch beam of the following dimensions: $h = 6$ inches; $A = 4$ inches; $I = 22.5$

$$q = \frac{22.5}{0.5 \times 6^2 \times 4} = 0.312$$

$$A = \frac{2.5}{5} \left[\frac{1}{2} \left(\frac{1}{0.446} + 0.446 \right) + \frac{120 \times 0.895}{12 \times 0.312 \times 6} \right] = 3.06 \text{ ins.};$$

showing that the six-inch beam has a greater sectional area than required.

If the load is concentrated at the apex of roof, the compressive strain $C = 2.8$ tons, and the area necessary to resist this strain

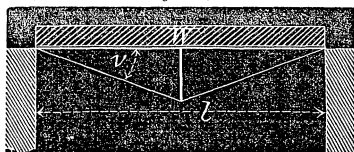
would be (taking p at five tons per square inch) $\frac{2.8}{5} = 0.56$ sq.

inches, provided this is able to resist buckling.

By comparing this with the above result, it will be seen how much greater the sectional area will have to be to resist a cross-breaking strain, caused by the load being distributed. These remarks also apply to simple trusses.

SIMPLE TRUSS, (BEAM CONTINUOUS OVER STRUT.)

Fig. 274.



EXAMPLE.

Reference.

$W = 20$ tons. $l = 20$ feet. $v = 15^\circ$ $p = 5$ tons per sq. inch.

We will assume a Phoenix Co's twelve-inch beam of the following dimensions:

$$\begin{aligned} h &= 12 \text{ inches.} & I &= 275.92 \\ A &= 12.5 \text{ inches.} & s &= 6 \text{ inches.} \end{aligned}$$

$$q = \frac{275.92}{0.5 \times 12^2 \times 12.5} = 0.306$$

$$m = 0.0703 \times 1 \times 120^2 = 84.36 \quad (\text{See Reaction of Supports.})$$

$$C = 23.32 \text{ tons.}$$

$$A = \frac{1}{5} \left(\frac{84.36}{0.306 \times 12} \right) + 23.32 = \frac{46.26}{5} = 9.25 \text{ inches.}$$

Consequently the sectional area of the twelve-inch beam is amply sufficient.

[NOTE.—The formulas for horizontal beams are also applicable to rafters of roof trusses, m and C being given. For the bending moments (m) the various distances are the horizontal projections of those on the rafter from abutment to ridge.

The foregoing formulas also apply to beams under a cross-breaking and tensional strain. If the truss (Fig. 274) is inverted, the horizontal member will be in tension. Hence, insert the resistance of the material to tension instead of compression, and put tensional for compressive strain; otherwise, the formulas remain the same.]

WEIGHT OF MOVING LOADS.

Variable and Accidental Loads.

(Weight of construction not included.)

Character of structure.	How loaded.	Weight in lbs. per square foot of surface.	
Street bridges for horse cars and heavy traffic.	Crow'd with persons.	Minimum.....	40 lbs.
		Maximum.....	120 "
		Average.....	80 "
Street bridges for general traffic, foot passengers, &c.	Persons, animals, and wagons.	Public travel....	80 lbs.
		Private travel ..	40 "
		Heavy business wagons.....	80 "
		Light business wagons.....	40 "
Floors, &c.....	Crowded public places.	Minimum.....	40 lbs.
		Maximum.....	120 "
		Average.....	80 "
	Dwellings.....	40 "
	Churches, court-rooms, theatres, and ball-rooms.	80 "
	Storage of grain...	100 "
	General merchandise.....	200 "
	Warehouses.....	250 "
	Factories.....	200 to 400 "
	Hay-lofts.....	80 "

STATIC AND MOVING LOADS ON BRIDGES OF WROUGHT IRON.

The following table gives an approximate weight per lineal foot in pounds of the static load or weight of construction complete for *Single-Line Railway Bridges*, supported at the ends, from ten to four hundred feet span; also the weight of the moving load per lineal foot of span, based on the assumption that the heaviest locomotives exert a pressure of three thousand pounds per lineal foot between their extreme bearings.

The table is applicable in computing the strains in all trusses with parallel booms mentioned in this work.

Weight of Construction and Moving Load of Wrought-Iron Single-Line Railway Bridges for the heaviest traffic.

(From 20 to 400 feet span.)

Weight of construction complete, including cross-ties and rails.				Weight of moving load equal to 3,000 lbs. per lineal foot of load.			
Span in ft.	Weight in lbs. per lineal foot of span.	Span in ft.	Weight in lbs. per lineal foot of span.	Span in ft.	Weight in lbs. per lineal foot of span.	Span in ft.	Weight in lbs. per lin. foot of span.
10	427	210	1,891	10	6,300	210	2,535
20	500	220	1,964	20	5,370	220	2,495
30	573	230	2,037	30	4,250	230	2,455
40	646	240	2,110	40	3,780	240	2,375
50	719	250	2,183	50	3,550	250	2,335
60	792	260	2,256	60	3,400	260	2,290
70	865	270	2,329	70	3,300	270	2,245
80	938	280	2,402	80	3,250	280	2,200
90	1,011	290	2,475	90	3,180	290	2,160
100	1,084	300	2,548	100	3,120	300	2,120
110	1,157	310	2,621	110	3,050	310	2,080
120	1,230	320	2,694	120	3,000	320	2,045
130	1,303	330	2,767	130	2,930	330	2,010
140	1,380	340	2,840	140	2,880	340	1,975
150	1,453	350	2,913	150	2,820	350	1,940
160	1,526	360	2,986	160	2,760	360	1,910
170	1,599	370	3,059	170	2,700	370	1,880
180	1,672	380	3,132	180	2,655	380	1,850
190	1,745	390	3,205	190	2,615	390	1,820
200	1,818	400	3,278	200	2,575	400	1,800

The following gives the actual weight of some well-known Bridges (single line) in America, Germany, and England:

Name of Bridge.	System.	Span in feet.	Weight of construction per lineal foot.	Weight of moving load per lineal foot.	Strain in boom per square inch.
			Lbs.	Lbs.	Lbs.
"Brenz," near Königsbronn...	{ Open Web, parallel booms. }	63.0	760	3,131	7,530
"Colomak".....	"	111.0	1,090	3,067	9,516
"Iser," near Munich	"	164.7	1,770	3,656	8,532
"Donau," near Ingolstadt.....	"	178.0	1,954	3,312	8,532
"Elb," near Meissen.....	"	179.0	1,324	2,783	10,390
"Rhine," near Mainz.....	{ "Pauli's," parabolic arched booms. }	345.0	2,170	1,970	11,660
"Royal Albert," near Saltash...	"	455.0	4,418	2,240	9,954
"Boyne".....	Lattice.....	264.0	3,225		
"Leven".....	"	36.0	566		
"Kent".....	"	36.0	580		
"Harper's Ferry"	Truss.....	124.0	770		

MISCELLANEOUS.

GEOMETRY.

LONGIMETRY AND PLANIMETRY.

(Lines and Areas.)

Reference.

A = Area.

π = Periphery of circle = 3.14159 when diameter = 1.

r = Radius of circle.

c = Length of cord of segment.

p = Circumference of circle for given diameter.

l = Length of circle arc, &c.

h = Height of segment.

v = Angles, expressed in decimals, as $15^{\circ} 30' = 15.5$.

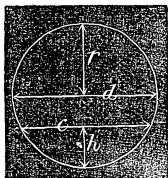
For other designations, see Figures.

[NOTE.—Always use the same unit for dimensions.]

Values of π .

$\pi = 1.14159$	$\frac{\pi}{3} = 1.04720$
$2\pi = 6.28319$	$\frac{\pi}{4} = 0.78540$
$\frac{1}{\pi} = 0.31831$	$\frac{\pi}{6} = 0.52360$
$\frac{1}{2\pi} = 0.15915$	$\pi^2 = 9.86960$
$\frac{1}{\pi^2} = 0.10132$	$\pi^3 = 31.00628$
$\frac{2}{\pi} = 0.63662$	$\sqrt{\pi} = 1.77245$
$\frac{\pi}{2} = 1.57080$	$\sqrt[3]{\pi} = 1.46459$
	$\sqrt{\frac{1}{\pi}} = 0.56419$

Fig. 275.

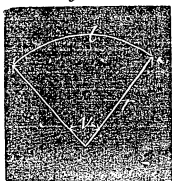


$$p = \pi d$$

$$d = \frac{p}{\pi}$$

$$r = \frac{p}{2\pi}$$

Fig. 276.

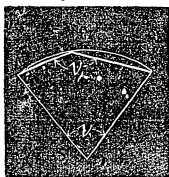


$$l = \frac{v}{360^\circ} p = \frac{v\pi d}{360^\circ} = \frac{v\pi r}{180^\circ}$$

$$v = \frac{l}{\pi r} 180^\circ$$

$$r = \frac{180^\circ}{v} \frac{l}{\pi}$$

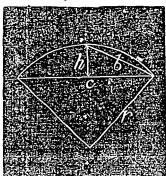
Fig. 277.



$$v_1 = 180^\circ - \frac{v}{2}$$

$$v = 2(180^\circ - v_1)$$

Fig. 278.

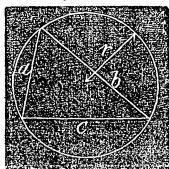


$$r = \frac{c^2 + 4h^2}{8h} = \frac{b^2}{2h}$$

$$c = 2\sqrt{2hr - h^2}$$

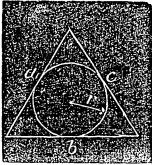
$$h = r - \sqrt{r^2 - \left(\frac{c}{2}\right)^2}$$

Fig. 279.



$$r = \frac{ac}{2\sqrt{a^2 - \left(\frac{a^2 + b^2 - c^2}{2b}\right)^2}}$$

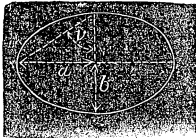
Fig. 280.



$$r = \frac{b \sqrt{a^2 - \left(\frac{a^2 + b^2 - c^2}{2b} \right)^2}}{a + b + c}$$

Fig. 281.

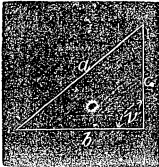
Ellipse.



$$p = (a + b) \left[1 + \frac{n^2}{4} + \frac{n^4}{64} + \frac{n^6}{256} + \dots \right]$$

$$\text{When } n = \frac{a - b}{a + b}$$

Fig. 282.

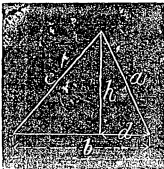


$$c = \sqrt{a^2 - b^2}$$

$$b = \sqrt{a^2 - c^2}$$

$$a = \sqrt{b^2 + c^2}$$

Fig. 283.

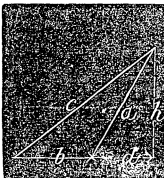


$$c^2 = a^2 + b^2 - 2bd$$

$$h = \sqrt{a^2 - d^2}$$

$$d = \frac{a^2 + b^2 - c^2}{2b}$$

Fig. 284.

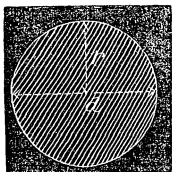


$$c^2 = a^2 + b^2 + 2bd$$

$$h^2 = \sqrt{a^2 - d^2}$$

$$d = \frac{c^2 - a^2 - b^2}{2b}$$

Fig. 285. (Circle plane.)

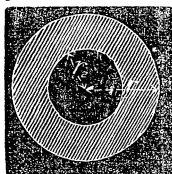


$$A = \pi r^2 = \frac{\pi d^2}{4} = 0.7854 d^2$$

$$r = \sqrt{\frac{A}{\pi}} = 0.5642 \sqrt{A}$$

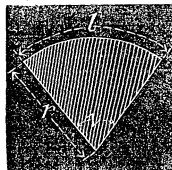
$$d = \sqrt{\frac{4A}{\pi}} = 1.1284 \sqrt{A}$$

Fig. 286. (Circle ring.)



$$A = \pi(r_1^2 - r_2^2) \\ = \pi(r_1 + r_2)(r_1 - r_2)$$

Fig. 287. (Sector.)

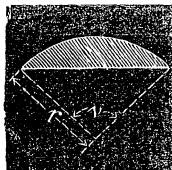


$$A = \frac{1}{2} l r = \frac{1}{2} v r^2 = \frac{v}{360^\circ} \pi r^2$$

$$= 0.008727 v r^2. \quad v = \frac{A}{\pi r^2} 360^\circ$$

$$r = \sqrt{\frac{360^\circ}{v} \frac{A}{\pi}} = \sqrt{\frac{2A}{v}}$$

Fig. 288. (Segment.)

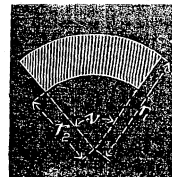


$$A = (v - \sin. v) \frac{r^2}{2}$$

$$= \left(\frac{v\pi}{180^\circ} - \sin. v \right) \frac{r^2}{2}$$

$$= (0.017453 v - \sin. v) \frac{r^2}{2}$$

Fig. 289. (Circled ring sector)

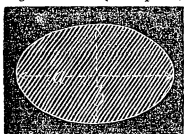


$$A = \frac{v(r_1^2 - r_2^2)}{2}$$

$$= \frac{v\pi}{360^\circ} (r_1^2 - r_2^2)$$

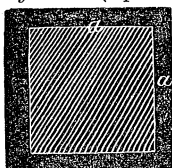
$$= 0.008727 v (r_1^2 - r_2^2)$$

Fig. 290. (Ellipse.)



$$A = \pi ab$$

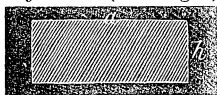
Fig. 291. (Square.)



$$A = a^2$$

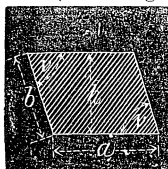
$$a = \sqrt{A}$$

Fig. 292. (Rectangle.)



$$A = ah$$

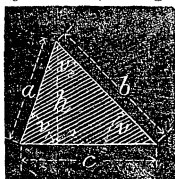
Fig. 293. (Parallelogram.)



$$A = ab \sin. v$$

$$= ah$$

Fig. 294. (Triangle.)



$$A = \frac{ch}{2} = \frac{1}{2} bc \sin. v$$

$$= \frac{c^2 \sin. v \sin. v_1}{2 \sin. v_2}$$

When the three sides are given:

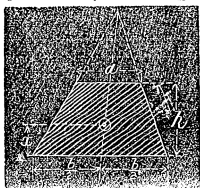
Let $a + b + c = s$

$$A = \sqrt{\frac{1}{2}s \left(\frac{1}{2}s - a \right) \left(\frac{1}{2}s - b \right) \left(\frac{1}{2}s - c \right)}$$

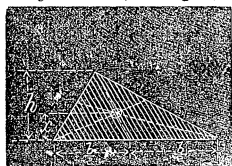
CENTER OF GRAVITY OF PLANES.

Reference.

- x = Distance from a fixed base to center of gravity.
 r = Radius.
 c = Chord.
 b, p, h = Dimensions.
 A = Area.
 v = Angle.

Fig. 295. (Quadrangle.) a and b parallel.

$$x = \frac{h}{2} - \frac{h}{6} \left(\frac{b-a}{b+a} \right)$$

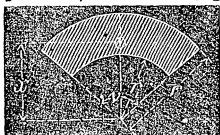
Fig. 296. (Triangle.)

$$x = \frac{h}{3}$$

Fig. 297. (Half circle, or elliptic plane.)

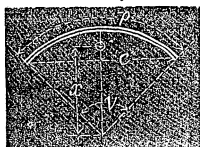
$$\frac{b}{2} = \text{radius} = r$$

$$x = 0.4244r$$

Fig. 298. (Concentric ring.)

$$x = \frac{4}{3} \frac{\sin. \frac{1}{2}v}{v} \frac{r^3 - r_1^3}{r^2 - r_1^2}$$

Fig. 299. (Circle, or elliptic arc.)



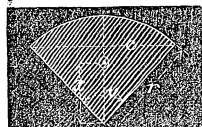
$$x = \frac{rc}{p} \frac{2 \sin. \frac{1}{2} v}{v} r$$

Fig. 300. (Half circumference of circle or ellipse.)



$$x = \frac{2}{\pi} r = 0.6366 r$$

Fig. 301. (Circle sector.)



$$x = \frac{4}{3} \frac{\sin. \frac{1}{2} v^2}{v} r$$

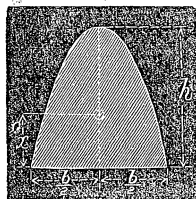
Fig. 302. (Circle segment.)



$A = \text{Area.}$

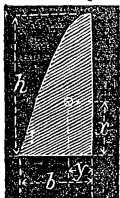
$$x = \frac{c^3}{12A}$$

Fig. 303. (Parabola.)



$$x = \frac{2h}{5}$$

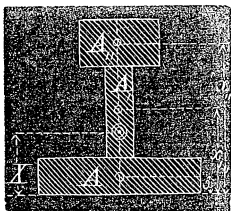
Fig. 305. (Half parabola.)



$$x = \frac{2}{5} h$$

$$y = \frac{3}{8} h$$

Fig. 305.



Of any section, composed of any number of simple figures:

Additional Reference.

A, A', A'' = Sectional area of simple figures.

X = Distance from center of gravity of whole section to axis mn .

x, x', x'' = Distance from center of gravity of a simple figure to a fixed axis mn .

$$X = \frac{Ax + A'x' + A''x'' + \&c.}{A + A' + A'' + \&c.}$$

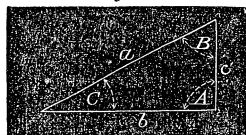
TRIGONOMETRICAL FORMULAS.

Reference.

a, b, c = Length of sides.
 A, B, C = Angles opposite to a, b, c respectively.

Right Angle Triangle.

Fig. 306.



$$A = 90^\circ$$

$$a = \sqrt{b^2 + c^2}$$

$$a = \frac{c}{\sin. C}$$

$$a = \frac{b}{\cos. C}$$

$$\text{Tang. } C = \frac{c}{b} = \frac{\sin. C}{\cos. C} = \frac{1}{\cot. C}$$

$$b = a \cos. C$$

$$b = c \cot. C$$

$$b = a \sin. B$$

$$b = c \text{ tang. } B$$

$$c = b \text{ tang. } C$$

$$c = a \sin. C$$

$$\text{Cotang. } C = \frac{\cos. C}{\sin. C} = \frac{1}{\text{tang. } C}$$

$$\text{Secant } C = \frac{1}{\cos. C}$$

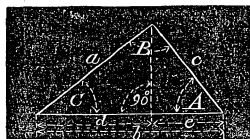
$$\sin. C = \frac{c}{a}$$

$$\text{Cosec. } C = \frac{1}{\sin. C}$$

$$\cos. C = \frac{b}{a}$$

Oblique Angle Triangle.

Fig. 307.



$$a = \frac{c \sin. A}{\sin. C}$$

$$a = \frac{c \sin. A}{\sin. (A + B)}$$

$$a = \sqrt{b^2 + c^2 - 2bc \cos. A}$$

$$b = \frac{c \sin. B}{\sin. C}$$

$$c = \frac{1}{2} \left(b - \frac{(a+c)(a-c)}{b} \right)$$

$$d = \frac{1}{2} \left(b + \frac{(a+c)(a-c)}{b} \right)$$

$$\sin. C = \frac{c \sin. B}{b} = \frac{c \sin. A}{a}$$

$$\sin. A = \frac{a \sin. C}{c}$$

NATURAL SINE

Deg.	Minutes.						
	0	5	10	15	20	25	30
0	.00000	.00145	.00291	.00436	.00582	.00727	.00873
1	.01745	.01891	.02036	.02181	.02327	.02472	.02618
2	.03490	.03635	.03781	.03926	.04071	.04217	.04362
3	.05234	.05379	.05524	.05669	.05814	.05960	.06105
4	.06976	.07121	.07266	.07411	.07556	.07701	.07846
5	.08716	.08860	.09005	.09150	.09295	.09440	.09585
6	.10453	.10597	.10742	.10887	.11031	.11176	.11320
7	.12187	.12331	.12476	.12620	.12764	.12908	.13053
8	.13917	.14061	.14205	.14349	.14493	.14637	.14781
9	.15643	.15787	.15931	.16074	.16218	.16361	.16505
10	.17365	.17508	.17651	.17794	.17937	.18081	.18224
11	.19081	.19224	.19366	.19509	.19652	.19794	.19937
12	.20791	.20933	.21076	.21218	.21360	.21502	.21644
13	.22495	.22637	.22778	.22920	.23062	.23203	.23345
14	.24192	.24333	.24474	.24615	.24756	.24897	.25038
15	.25882	.26022	.26163	.26303	.26443	.26584	.26724
16	.27564	.27704	.27843	.27983	.28123	.28262	.28402
17	.29237	.29376	.29515	.29654	.29793	.29932	.30071
18	.30902	.31040	.31178	.31316	.31454	.31593	.31730
19	.32557	.32694	.32832	.32969	.33106	.33244	.33381
20	.34202	.34339	.34475	.34612	.34748	.34884	.35021
21	.35837	.35973	.36108	.36244	.36379	.36515	.36650
22	.37461	.37595	.37730	.37865	.37999	.38134	.38268
23	.39073	.39207	.39341	.39474	.39608	.39741	.39875
24	.40674	.40806	.40939	.41072	.41204	.41337	.41469
25	.42232	.42364	.42495	.42627	.42758	.42890	.43021
26	.43837	.43968	.44098	.44229	.44359	.44489	.44620
27	.45399	.45529	.45658	.45787	.45917	.46046	.46175
28	.46947	.47076	.47204	.47332	.47460	.47588	.47716
29	.48481	.48608	.48735	.48862	.48989	.49116	.49242
30	.50000	.50126	.50252	.50377	.50503	.50628	.50754
31	.51504	.51628	.51753	.51877	.52002	.52126	.52250
32	.52992	.53115	.53238	.53361	.53484	.53607	.53730
33	.54464	.54586	.54708	.54829	.54951	.55072	.55194
34	.55919	.56040	.56160	.56280	.56401	.56521	.56641
35	.57358	.57477	.57596	.57715	.57833	.57952	.58070
36	.58779	.58897	.59014	.59131	.59248	.59365	.59482
37	.60182	.60298	.60414	.60529	.60645	.60761	.60876
38	.61566	.61681	.61795	.61909	.62024	.62138	.62251
39	.62932	.63045	.63158	.63271	.63383	.63496	.63608
40	.64279	.64390	.64501	.64612	.64723	.64834	.64945
41	.65606	.65716	.65825	.65935	.66044	.66153	.66262
42	.66913	.67021	.67129	.67237	.67344	.67452	.67559
43	.68200	.68306	.68412	.68518	.68624	.68730	.68835
44	.69466	.69570	.69675	.69779	.69883	.69987	.70091
Deg.	Minutes.						
	60	55	50	45	40	35	30

NATURAL COSINE.

NATURAL SINE.

Minutes.						Deg.
35	40	45	50	55	60	
.01018	.01164	.01309	.01454	.01600	.01745	89
.02763	.02908	.03054	.03199	.03345	.03490	88
.04507	.04653	.04798	.04943	.05088	.05234	87
.06250	.06395	.06540	.06685	.06831	.06976	86
.07991	.08136	.08281	.08426	.08571	.08716	85
.09729	.09874	.10019	.10164	.10308	.10453	84
.11465	.11609	.11754	.11898	.12043	.12187	83
.13197	.13341	.13485	.13629	.13802	.13917	82
.14925	.15069	.15212	.15356	.15500	.15643	81
.16648	.16792	.16935	.17078	.17222	.17365	80
.18367	.18509	.18652	.18795	.18938	.19081	79
.20079	.20222	.20364	.20507	.20649	.20791	78
.21786	.21928	.22070	.22212	.22353	.22495	77
.23486	.23627	.23769	.23910	.24051	.24192	76
.25179	.25320	.25460	.25601	.25741	.25882	75
.26864	.27004	.27144	.27284	.27421	.27564	74
.28541	.28680	.28820	.28959	.29098	.29237	73
.30209	.30348	.30486	.30625	.30763	.30902	72
.31868	.32006	.32144	.32282	.32419	.32557	71
.33518	.33655	.33792	.33929	.34065	.34202	70
.35157	.35293	.35429	.35565	.35701	.35837	69
.36785	.36921	.37056	.37191	.37326	.37461	68
.38403	.38537	.38671	.38805	.38939	.39073	67
.40008	.40141	.40275	.40408	.40541	.40674	66
.41602	.41734	.41866	.41998	.42130	.42262	65
.43182	.43313	.43445	.43575	.43706	.43837	64
.44750	.44880	.45010	.45140	.45269	.45399	63
.46304	.46433	.46561	.46690	.46819	.46947	62
.47844	.47971	.48099	.48226	.48354	.48481	61
.49369	.49495	.49622	.49748	.49874	.50000	60
.50879	.51004	.51129	.51254	.51379	.51504	59
.52374	.52498	.52621	.52745	.52869	.52992	58
.53853	.53975	.54097	.54220	.54342	.54464	57
.55315	.55436	.55557	.55678	.55799	.55919	56
.56760	.56880	.57000	.57119	.57238	.57358	55
.58189	.58307	.58425	.58543	.58661	.58779	54
.59599	.59716	.59832	.59949	.60065	.60182	53
.60991	.61107	.61222	.61337	.61451	.61566	52
.62365	.62479	.62595	.62706	.62819	.62932	51
.63720	.63832	.63944	.64056	.64167	.64279	50
.65055	.65166	.65276	.65386	.65496	.65606	49
.66371	.66480	.66588	.66697	.66805	.66913	48
.67666	.67773	.67880	.67987	.68093	.68200	47
.68941	.69046	.69151	.69256	.69361	.69466	46
.70195	.70298	.70401	.70505	.70608	.70711	45
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL CO-SINE.

NATURAL SINE.

Deg.	Minutes.						
	0	5	10	15	20	25	30
45	.70711	.70813	.70916	.71019	.71121	.71223	.71325
46	.71934	.72035	.72136	.72236	.72337	.72437	.72537
47	.73135	.73234	.73333	.73432	.73531	.73629	.73728
48	.74314	.74412	.74509	.74606	.74703	.74799	.74896
49	.75471	.75566	.75661	.75756	.75851	.75946	.76041
50	.76604	.76698	.76791	.76884	.76977	.77070	.77162
51	.77715	.77806	.77897	.77988	.78079	.78170	.78261
52	.78801	.78891	.78980	.79069	.79158	.79247	.79335
53	.79864	.79951	.80038	.80125	.80212	.80299	.80386
54	.80902	.80987	.81072	.81157	.81242	.81327	.81412
55	.81915	.81999	.82082	.82165	.82248	.82330	.82413
56	.82904	.82985	.83066	.83147	.83228	.83308	.83389
57	.83867	.83946	.84025	.84104	.84182	.84261	.84339
58	.84805	.84882	.84959	.85035	.85112	.85188	.85264
59	.85717	.85792	.85866	.85941	.86015	.86089	.86163
60	.86603	.86675	.86748	.86820	.86892	.86964	.87036
61	.87462	.87532	.87603	.87673	.87743	.87812	.87882
62	.88295	.88363	.88431	.88499	.88566	.88634	.88701
63	.89101	.89167	.89232	.89298	.89363	.89428	.89493
64	.89879	.89943	.90007	.90070	.90133	.90196	.90259
65	.90631	.90692	.90753	.90814	.90875	.90936	.90996
66	.91355	.91414	.91472	.91531	.91590	.91648	.91706
67	.92350	.92407	.92464	.92520	.92576	.92632	.92688
68	.92718	.92773	.92827	.92881	.92935	.92988	.93042
69	.93358	.93410	.93462	.93514	.93565	.93616	.93667
70	.93969	.94019	.94068	.94118	.94167	.94215	.94264
71	.94552	.94599	.94646	.94693	.94740	.94786	.94832
72	.95106	.95150	.95191	.95240	.95284	.95328	.95372
73	.95630	.95673	.95715	.95757	.95799	.95841	.95882
74	.96126	.96166	.96206	.96246	.96285	.96324	.96363
75	.96593	.96630	.96667	.96705	.96742	.96778	.96815
76	.97030	.97065	.97100	.97134	.97169	.97203	.97237
77	.97437	.97470	.97502	.97534	.97566	.97598	.97630
78	.97815	.97845	.97875	.97905	.97934	.97963	.97992
79	.98163	.98190	.98218	.98245	.98272	.98299	.98325
80	.98481	.98506	.98531	.98556	.98580	.98604	.98629
81	.98769	.98791	.98814	.98836	.98858	.98880	.98902
82	.99027	.99047	.99067	.99087	.99106	.99125	.99144
83	.99255	.99272	.99290	.99307	.99324	.99341	.99357
84	.99452	.99467	.99482	.99497	.99511	.99526	.99540
85	.99619	.99632	.99644	.99657	.99668	.99680	.99692
86	.99756	.99766	.99776	.99786	.99795	.99804	.99813
87	.99863	.99870	.99878	.99885	.99892	.99898	.99905
88	.99939	.99944	.99949	.99953	.99958	.99962	.99966
89	.99985	.99987	.99989	.99991	.99993	.99995	.99996
Deg.	60	55	50	45	40	35	30
Minutes.							

NATURAL COSINE.

NATURAL SINE.

Minutes.						Deg.
35	40	45	50	55	60	
.71427	.71529	.71630	.71732	.71833	.71934	44
.72637	.72737	.72837	.72937	.73036	.73135	43
.73826	.73924	.74022	.74120	.74217	.74314	42
.74992	.75088	.75184	.75280	.75375	.75471	41
.76135	.76229	.76323	.76417	.76511	.76604	40
.77255	.77347	.77439	.77531	.77623	.77715	39
.78351	.78442	.78532	.78622	.78711	.78801	38
.79424	.79512	.79600	.79688	.79776	.79864	37
.80472	.80558	.80644	.80730	.80816	.80902	36
.81496	.81580	.81664	.81748	.81832	.81915	35
.82495	.82577	.82659	.82741	.82822	.82904	34
.83469	.83549	.83629	.83708	.83788	.83867	33
.84417	.84495	.84573	.84650	.84728	.84805	32
.85340	.85416	.85491	.85567	.85642	.85717	31
.86237	.86317	.86384	.86457	.86530	.86603	30
.87107	.87178	.87250	.87321	.87391	.87462	29
.87959	.88020	.88089	.88158	.88226	.88295	28
.88768	.88835	.88902	.88968	.89035	.89101	27
.89558	.89623	.89687	.89752	.89816	.89879	26
.90321	.90383	.90446	.90507	.90569	.90631	25
.91056	.91116	.91176	.91236	.91295	.91355	24
.91764	.91822	.91879	.91936	.91994	.92050	23
.92444	.92499	.92554	.92609	.92664	.92718	22
.93095	.93148	.93201	.93253	.93306	.93358	21
.93718	.93769	.93819	.93869	.93919	.93969	20
.94313	.94361	.94409	.94457	.94504	.94552	19
.94878	.94924	.94970	.95015	.95061	.95106	18
.95415	.95459	.95502	.95545	.95588	.95630	17
.95923	.95964	.96005	.96046	.96086	.96126	16
.96402	.96440	.96479	.96517	.96555	.96593	15
.96851	.96887	.96923	.96959	.96994	.97030	14
.97271	.97304	.97338	.97371	.97404	.97437	13
.97661	.97692	.97723	.97754	.97784	.97815	12
.98021	.98050	.98079	.98107	.98135	.98163	11
.98352	.98378	.98404	.98430	.98455	.98481	10
.98652	.98676	.98700	.98723	.98746	.98769	9
.98923	.98944	.98965	.98986	.99006	.99027	8
.99163	.99182	.99200	.99219	.99237	.99255	7
.99374	.99390	.99406	.99421	.99437	.99452	6
.99553	.99567	.99580	.99594	.99607	.99619	5
.99703	.99714	.99725	.99736	.99746	.99756	4
.99822	.99831	.99839	.99847	.99855	.99863	3
.99911	.99917	.99923	.99929	.99934	.99939	2
.99969	.99973	.99976	.99979	.99982	.99985	1
.99997	.99998	.99999	1.00000	1.00000	1.00000	0
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL COSINE.

NATURAL TANGENT.

Deg.	Minutes.						
	0	5	10	15	20	25	30
0	0.0000	0.0014	0.0029	0.0044	0.0058	0.0073	0.0087
1	0.0175	0.0189	0.0204	0.0218	0.0233	0.0247	0.0262
2	0.0349	0.0364	0.0378	0.0393	0.0407	0.0422	0.0437
3	0.0524	0.0539	0.0553	0.0568	0.0582	0.0597	0.0612
4	0.0699	0.0714	0.0728	0.0743	0.0758	0.0772	0.0787
5	0.0875	0.0889	0.0904	0.0919	0.0933	0.0948	0.0963
6	0.1051	0.1066	0.1080	0.1095	0.1110	0.1125	0.1139
7	0.1228	0.1243	0.1257	0.1272	0.1287	0.1302	0.1316
8	0.1405	0.1420	0.1435	0.1450	0.1465	0.1480	0.1495
9	0.1584	0.1599	0.1614	0.1629	0.1644	0.1658	0.1673
10	0.1763	0.1778	0.1793	0.1808	0.1823	0.1838	0.1853
11	0.1944	0.1959	0.1974	0.1989	0.2004	0.2019	0.2034
12	0.2126	0.2141	0.2156	0.2171	0.2186	0.2202	0.2217
13	0.2309	0.2324	0.2339	0.2355	0.2370	0.2385	0.2401
14	0.2493	0.2509	0.2524	0.2540	0.2555	0.2571	0.2586
15	0.2679	0.2695	0.2711	0.2726	0.2742	0.2758	0.2773
16	0.2867	0.2883	0.2899	0.2915	0.2930	0.2946	0.2962
17	0.3057	0.3073	0.3089	0.3105	0.3121	0.3137	0.3153
18	0.3249	0.3265	0.3281	0.3297	0.3314	0.3330	0.3346
19	0.3443	0.3460	0.3476	0.3492	0.3508	0.3525	0.3541
20	0.3640	0.3656	0.3673	0.3689	0.3706	0.3722	0.3739
21	0.3839	0.3855	0.3872	0.3889	0.3905	0.3922	0.3939
22	0.4040	0.4057	0.4074	0.4091	0.4108	0.4125	0.4142
23	0.4245	0.4262	0.4279	0.4296	0.4314	0.4331	0.4348
24	0.4452	0.4470	0.4487	0.4505	0.4522	0.4540	0.4557
25	0.4663	0.4681	0.4698	0.4716	0.4734	0.4752	0.4770
26	0.4877	0.4895	0.4913	0.4931	0.4950	0.4968	0.4986
27	0.5095	0.5114	0.5132	0.5150	0.5169	0.5187	0.5206
28	0.5317	0.5336	0.5354	0.5373	0.5392	0.5411	0.5430
29	0.5543	0.5562	0.5581	0.5600	0.5619	0.5638	0.5658
30	0.5774	0.5793	0.5812	0.5832	0.5851	0.5871	0.5891
31	0.6008	0.6028	0.6048	0.6068	0.6088	0.6108	0.6128
32	0.6249	0.6269	0.6289	0.6309	0.6330	0.6350	0.6371
33	0.6494	0.6515	0.6535	0.6556	0.6577	0.6598	0.6619
34	0.6745	0.6766	0.6787	0.6809	0.6830	0.6851	0.6873
35	0.7002	0.7024	0.7045	0.7067	0.7089	0.7111	0.7133
36	0.7265	0.7288	0.7310	0.7332	0.7355	0.7377	0.7400
37	0.7536	0.7558	0.7581	0.7604	0.7627	0.7650	0.7673
38	0.7813	0.7836	0.7860	0.7883	0.7907	0.7931	0.7954
39	0.8098	0.8122	0.8146	0.8170	0.8195	0.8219	0.8243
40	0.8391	0.8416	0.8441	0.8466	0.8491	0.8516	0.8541
41	0.8693	0.8718	0.8744	0.8770	0.8795	0.8821	0.8847
42	0.9004	0.9030	0.9057	0.9083	0.9110	0.9137	0.9163
43	0.9325	0.9352	0.9380	0.9407	0.9434	0.9462	0.9490
44	0.9657	0.9685	0.9713	0.9742	0.9770	0.9798	0.9827
Deg.	60	55	50	45	40	35	30
Minutes.							

NATURAL COTANGENT.

NATURAL TANGENT.

Minutes.						Deg.
35	40	45	50	55	60	
0.0102	0.0116	0.0131	0.0145	0.0160	0.0175	89
0.0276	0.0291	0.0305	0.0320	0.0335	0.0349	88
0.0451	0.0466	0.0480	0.0495	0.0509	0.0524	87
0.0626	0.0641	0.0655	0.0670	0.0685	0.0699	86
0.0802	0.0816	0.0831	0.0846	0.0860	0.0875	85
0.0978	0.0992	0.1007	0.1022	0.1036	0.1051	84
0.1154	0.1169	0.1184	0.1198	0.1213	0.1228	83
0.1331	0.1346	0.1361	0.1376	0.1391	0.1405	82
0.1509	0.1524	0.1539	0.1554	0.1569	0.1584	81
0.1688	0.1703	0.1718	0.1733	0.1748	0.1763	80
0.1868	0.1883	0.1899	0.1914	0.1929	0.1944	79
0.2050	0.2065	0.2080	0.2095	0.2110	0.2126	78
0.2232	0.2247	0.2263	0.2278	0.2293	0.2309	77
0.2416	0.2432	0.2447	0.2462	0.2478	0.2493	76
0.2602	0.2617	0.2633	0.2648	0.2664	0.2679	75
0.2789	0.2805	0.2820	0.2836	0.2852	0.2867	74
0.2978	0.2994	0.3010	0.3026	0.3041	0.3057	73
0.3169	0.3185	0.3201	0.3217	0.3233	0.3249	72
0.3362	0.3378	0.3394	0.3411	0.3427	0.3443	71
0.3558	0.3574	0.3590	0.3607	0.3623	0.3640	70
0.3755	0.3772	0.3789	0.3805	0.3822	0.3839	69
0.3956	0.3973	0.3990	0.4006	0.4023	0.4040	68
0.4159	0.4176	0.4193	0.4210	0.4228	0.4245	67
0.4365	0.4383	0.4400	0.4417	0.4435	0.4452	66
0.4575	0.4592	0.4610	0.4628	0.4645	0.4663	65
0.4788	0.4805	0.4823	0.4841	0.4859	0.4877	64
0.5004	0.5022	0.5040	0.5059	0.5077	0.5095	63
0.5224	0.5243	0.5261	0.5280	0.5298	0.5317	62
0.5448	0.5467	0.5486	0.5505	0.5524	0.5543	61
0.5677	0.5696	0.5715	0.5735	0.5754	0.5774	60
0.5910	0.5930	0.5949	0.5969	0.5989	0.6008	59
0.6148	0.6168	0.6188	0.6208	0.6228	0.6249	58
0.6391	0.6412	0.6432	0.6453	0.6473	0.6494	57
0.6640	0.6661	0.6682	0.6703	0.6724	0.6745	56
0.6894	0.6916	0.6937	0.6959	0.6980	0.7002	55
0.7155	0.7177	0.7199	0.7221	0.7243	0.7265	54
0.7422	0.7445	0.7467	0.7490	0.7513	0.7536	53
0.7696	0.7720	0.7743	0.7766	0.7789	0.7813	52
0.7978	0.8002	0.8026	0.8050	0.8074	0.8098	51
0.8268	0.8292	0.8317	0.8341	0.8366	0.8391	50
0.8566	0.8591	0.8617	0.8642	0.8667	0.8693	49
0.8873	0.8899	0.8925	0.8951	0.8978	0.9004	48
0.9190	0.9217	0.9244	0.9271	0.9298	0.9325	47
0.9517	0.9545	0.9573	0.9601	0.9629	0.9657	46
0.9856	0.9884	0.9913	0.9942	0.9971	1.0000	45
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL COTANGENT.

NATURAL TANGENT.

Deg.	Minutes.						
	0	5	10	15	20	25	30
45	1.0000	1.0029	1.0058	1.0088	1.0117	1.0146	1.0176
46	1.0355	1.0385	1.0416	1.0446	1.0477	1.0507	1.0538
47	1.0724	1.0755	1.0786	1.0818	1.0850	1.0881	1.0913
48	1.1106	1.1139	1.1171	1.1204	1.1237	1.1270	1.1303
49	1.1504	1.1537	1.1571	1.1606	1.1640	1.1674	1.1708
50	1.1917	1.1953	1.1988	1.2024	1.2059	1.2095	1.2131
51	1.2349	1.2386	1.2423	1.2460	1.2497	1.2534	1.2572
52	1.2799	1.2838	1.2876	1.2915	1.2954	1.2993	1.3032
53	1.3270	1.3311	1.3351	1.3392	1.3432	1.3472	1.3514
54	1.3764	1.3806	1.3848	1.3891	1.3934	1.3976	1.4019
55	1.4281	1.4326	1.4370	1.4415	1.4460	1.4505	1.4550
56	1.4826	1.4872	1.4919	1.4966	1.5013	1.5061	1.5108
57	1.5399	1.5448	1.5497	1.5547	1.5597	1.5647	1.5697
58	1.6003	1.6055	1.6107	1.6160	1.6212	1.6265	1.6318
59	1.6643	1.6698	1.6753	1.6808	1.6864	1.6920	1.6976
60	1.7320	1.7379	1.7437	1.7496	1.7556	1.7615	1.7675
61	1.8040	1.8102	1.8165	1.8228	1.8291	1.8354	1.8418
62	1.8807	1.8873	1.8940	1.9007	1.9074	1.9142	1.9210
63	1.9626	1.9697	1.9768	1.9840	1.9912	1.9984	2.0057
64	2.0503	2.0579	2.0655	2.0732	2.0809	2.0887	2.0965
65	2.1445	2.1527	2.1609	2.1692	2.1775	2.1859	2.1943
66	2.2460	2.2549	2.2637	2.2727	2.2817	2.2907	2.2998
67	2.3558	2.3654	2.3750	2.3847	2.3945	2.4043	2.4142
68	2.4751	2.4855	2.4960	2.5065	2.5171	2.5279	2.5386
69	2.6051	2.6165	2.6279	2.6394	2.6511	2.6628	2.6746
70	2.7475	2.7600	2.7725	2.7852	2.7980	2.8109	2.8239
71	2.9042	2.9180	2.9319	2.9456	2.9600	2.9743	2.9886
72	3.0777	3.0930	3.1084	3.1240	3.1397	3.1556	3.1716
73	3.2708	3.2879	3.3052	3.3226	3.3402	3.3580	3.3759
74	3.4874	3.5067	3.5261	3.5457	3.5656	3.5856	3.6059
75	3.7320	3.7539	3.7760	3.7983	3.8208	3.8436	3.8667
76	4.0108	4.0358	4.0611	4.0867	4.1126	4.1388	4.1653
77	4.3315	4.3604	4.3897	4.4194	4.4494	4.4799	4.5107
78	4.7046	4.7385	4.7729	4.8077	4.8430	4.8788	4.9152
79	5.1445	5.1848	5.2257	5.2671	5.3093	5.3521	5.3955
80	5.6713	5.7199	5.7694	5.8197	5.8708	5.9228	5.9758
81	6.3137	6.3737	6.4348	6.4971	6.5605	6.6252	6.6912
82	7.1154	7.1912	7.2687	7.3479	7.4287	7.5113	7.5957
83	8.1443	8.2434	8.3450	8.4490	8.5555	8.6648	8.7769
84	9.5144	9.6493	9.7882	9.9310	10.0780	10.2290	10.3850
85	11.4300	11.6250	11.8260	12.0350	12.2510	12.4740	12.7060
86	14.5010	14.6060	14.9240	15.2570	15.6050	15.9690	16.3500
87	19.0810	19.6270	20.2060	20.8190	21.4700	22.1640	22.9040
88	28.6360	29.8820	31.2420	32.7300	34.3680	36.1780	38.1880
89	57.2900	62.4990	68.7500	76.3900	85.9480	98.2180	114.5900
Deg.	Minutes.						
	60	55	50	45	40	35	30

NATURAL COTANGENT.

NATURAL TANGENT.

Minutes.						Deg.
35	40	45	50	55	60	
1.0206	1.0235	1.0265	1.0295	1.0325	1.0355	44
1.0568	1.0590	1.0630	1.0661	1.0692	1.0724	43
1.0945	1.0977	1.1009	1.1041	1.1074	1.1106	42
1.1336	1.1369	1.1403	1.1436	1.1470	1.1504	41
1.1743	1.1778	1.1812	1.1847	1.1882	1.1917	40
1.2167	1.2203	1.2239	1.2276	1.2312	1.2349	39
1.2609	1.2647	1.2685	1.2723	1.2761	1.2799	38
1.3071	1.3111	1.3151	1.3190	1.3230	1.3270	37
1.3555	1.3597	1.3638	1.3680	1.3722	1.3764	36
1.4063	1.4106	1.4150	1.4193	1.4237	1.4281	35
1.4595	1.4641	1.4687	1.4733	1.4779	1.4826	34
1.5156	1.5204	1.5252	1.5301	1.5350	1.5399	33
1.5747	1.5798	1.5849	1.5900	1.5952	1.6003	32
1.6372	1.6426	1.6479	1.6534	1.6588	1.6643	31
1.7033	1.7090	1.7147	1.7205	1.7263	1.7320	30
1.7735	1.7795	1.7856	1.7917	1.7979	1.8040	29
1.8482	1.8546	1.8611	1.8676	1.8741	1.8807	28
1.9278	1.9347	1.9416	1.9486	1.9556	1.9626	27
2.0130	2.0204	2.0278	2.0353	2.0428	2.0503	26
2.1044	2.1123	2.1203	2.1283	2.1364	2.1445	25
2.2028	2.2113	2.2199	2.2286	2.2373	2.2460	24
2.3090	2.3183	2.3276	2.3369	2.3464	2.3558	23
2.4242	2.4342	2.4443	2.4545	2.4648	2.4751	22
2.5495	2.5605	2.5715	2.5826	2.5938	2.6051	21
2.6865	2.6985	2.7106	2.7228	2.7351	2.7475	20
2.8370	2.8502	2.8636	2.8770	2.8905	2.9042	19
3.0032	3.0178	3.0326	3.0475	3.0625	3.0777	18
3.1877	3.2041	3.2205	3.2371	3.2539	3.2708	17
3.3941	3.4124	3.4308	3.4495	3.4684	3.4874	16
3.6264	3.6471	3.6680	3.6891	3.7105	3.7320	15
3.8900	3.9136	3.9375	3.9616	3.9861	4.0108	14
4.1921	4.2193	4.2468	4.2747	4.3029	4.3315	13
4.5420	4.5736	4.6057	4.6382	4.6712	4.7046	12
4.9520	4.9894	5.0273	5.0658	5.1049	5.1445	11
5.4397	5.4845	5.5301	5.5764	5.6234	5.6713	10
6.0296	6.0844	6.1402	6.1970	6.2549	6.3137	9
6.7584	6.8269	6.8969	6.9682	7.0410	7.1154	8
7.6821	7.7703	7.8606	7.9530	8.0476	8.1443	7
8.8918	9.0098	9.1309	9.2553	9.3831	9.5144	6
10.5460	10.7120	10.8830	11.0590	11.2420	11.4300	5
12.9470	13.1970	13.4570	13.7270	14.0080	14.3010	4
16.7500	17.1690	17.6110	18.0750	18.5640	19.0810	3
23.6940	24.5420	25.4520	26.4320	27.4900	28.6360	2
40.4360	42.9640	45.8290	49.1040	52.8820	57.2900	1
137.5100	171.8800	229.1800	343.7700	687.5500		0
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL COTANGENT.

NATURAL SECANT.

Deg.	Minutes.						
	0	5	10	15	20	25	30
0	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000	1.0000
1	1.0001	1.0002	1.0002	1.0002	1.0003	1.0003	1.0003
2	1.0006	1.0007	1.0007	1.0008	1.0008	1.0009	1.0009
3	1.0014	1.0014	1.0015	1.0016	1.0017	1.0018	1.0019
4	1.0021	1.0025	1.0025	1.0027	1.0029	1.0030	1.0031
5	1.0038	1.0039	1.0041	1.0042	1.0043	1.0045	1.0046
6	1.0055	1.0057	1.0058	1.0060	1.0061	1.0063	1.0065
7	1.0075	1.0077	1.0079	1.0080	1.0082	1.0084	1.0086
8	1.0098	1.0100	1.0102	1.0104	1.0107	1.0109	1.0111
9	1.0125	1.0127	1.0129	1.0132	1.0134	1.0136	1.0139
10	1.0154	1.0157	1.0159	1.0162	1.0165	1.0167	1.0170
11	1.0187	1.0190	1.0193	1.0196	1.0199	1.0202	1.0205
12	1.0223	1.0223	1.0229	1.0233	1.0236	1.0239	1.0243
13	1.0263	1.0266	1.0270	1.0274	1.0277	1.0280	1.0284
14	1.0306	1.0310	1.0314	1.0317	1.0321	1.0325	1.0329
15	1.0353	1.0357	1.0361	1.0365	1.0369	1.0373	1.0377
16	1.0403	1.0407	1.0412	1.0416	1.0420	1.0425	1.0429
17	1.0457	1.0461	1.0466	1.0471	1.0476	1.0480	1.0485
18	1.0515	1.0520	1.0525	1.0530	1.0535	1.0540	1.0545
19	1.0577	1.0581	1.0587	1.0592	1.0598	1.0603	1.0608
20	1.0642	1.0647	1.0653	1.0659	1.0664	1.0670	1.0676
21	1.0711	1.0717	1.0723	1.0729	1.0736	1.0742	1.0748
22	1.0785	1.0792	1.0798	1.0804	1.0811	1.0817	1.0824
23	1.0864	1.0870	1.0877	1.0884	1.0891	1.0897	1.0904
24	1.0946	1.0953	1.0961	1.0968	1.0975	1.0982	1.0989
25	1.1034	1.1041	1.1049	1.1056	1.1064	1.1072	1.1079
26	1.1126	1.1134	1.1142	1.1150	1.1158	1.1166	1.1174
27	1.1223	1.1231	1.1240	1.1248	1.1257	1.1265	1.1274
28	1.1326	1.1334	1.1343	1.1352	1.1361	1.1370	1.1379
29	1.1433	1.1443	1.1452	1.1461	1.1471	1.1480	1.1489
30	1.1547	1.1557	1.1566	1.1576	1.1586	1.1596	1.1606
31	1.1666	1.1676	1.1687	1.1697	1.1707	1.1718	1.1728
32	1.1792	1.1802	1.1830	1.1824	1.1835	1.1846	1.1857
33	1.1923	1.1935	1.1946	1.1958	1.1969	1.1980	1.1992
34	1.2062	1.2074	1.2068	1.2098	1.2110	1.2122	1.2134
35	1.2208	1.2220	1.2233	1.2245	1.2258	1.2270	1.2283
36	1.2361	1.2374	1.2387	1.2400	1.2413	1.2427	1.2440
37	1.2521	1.2535	1.2549	1.2563	1.2577	1.2591	1.2605
38	1.2690	1.2705	1.2719	1.2734	1.2748	1.2763	1.2778
39	1.2867	1.2883	1.2898	1.2913	1.2929	1.2944	1.2960
40	1.3054	1.3070	1.3086	1.3102	1.3118	1.3134	1.3151
41	1.3250	1.3267	1.3284	1.3301	1.3318	1.3335	1.3352
42	1.3456	1.3474	1.3492	1.3509	1.3507	1.3540	1.3563
43	1.3673	1.3692	1.3710	1.3729	1.3748	1.3767	1.3786
44	1.3902	1.3921	1.3941	1.3960	1.3980	1.4000	1.4020
Deg.	60	55	50	45	40	35	30
Minutes.							

NATURAL COSECANT.

NATURAL SECANT.

Minutes.						Deg.
35	40	45	50	55	60	
1.0000	1.0001	1.0001	1.0001	1.0001	1.0001	89
1.0004	1.0004	1.0005	1.0005	1.0005	1.0006	88
1.0010	1.0011	1.0011	1.0012	1.0013	1.0014	87
1.0019	1.0020	1.0021	1.0022	1.0023	1.0024	86
1.0032	1.0033	1.0034	1.0036	1.0037	1.0038	85
1.0048	1.0049	1.0050	1.0052	1.0053	1.0055	84
1.0066	1.0068	1.0070	1.0071	1.0073	1.0075	83
1.0088	1.0090	1.0092	1.0094	1.0096	1.0098	82
1.0113	1.0115	1.0118	1.0120	1.0122	1.0125	81
1.0141	1.0145	1.0146	1.0149	1.0152	1.0154	80
1.0173	1.0176	1.0179	1.0181	1.0184	1.0187	79
1.0208	1.0211	1.0214	1.0217	1.0220	1.0223	78
1.0246	1.0249	1.0253	1.0256	1.0260	1.0263	77
1.0288	1.0291	1.0295	1.0298	1.0302	1.0306	76
1.0333	1.0337	1.0341	1.0345	1.0349	1.0353	75
1.0382	1.0386	1.0390	1.0394	1.0399	1.0403	74
1.0434	1.0438	1.0443	1.0448	1.0452	1.0457	73
1.0490	1.0495	1.0500	1.0505	1.0510	1.0515	72
1.0550	1.0555	1.0560	1.0565	1.0571	1.0577	71
1.0644	1.0649	1.0655	1.0660	1.0666	1.0672	70
1.0682	1.0688	1.0694	1.0699	1.0705	1.0711	69
1.0754	1.0760	1.0766	1.0773	1.0779	1.0785	68
1.0830	1.0837	1.0844	1.0850	1.0857	1.0864	67
1.0911	1.0918	1.0925	1.0932	1.0939	1.0946	66
1.0997	1.1004	1.1011	1.1019	1.1026	1.1034	65
1.1087	1.1095	1.1102	1.1110	1.1118	1.1126	64
1.1182	1.1190	1.1198	1.1207	1.1215	1.1223	63
1.1282	1.1291	1.1299	1.1308	1.1317	1.1326	62
1.1388	1.1397	1.1406	1.1415	1.1424	1.1433	61
1.1499	1.1508	1.1518	1.1528	1.1537	1.1547	60
1.1616	1.1626	1.1636	1.1646	1.1656	1.1666	59
1.1739	1.1749	1.1760	1.1770	1.1781	1.1792	58
1.1868	1.1879	1.1889	1.1901	1.1912	1.1923	57
1.2004	1.2015	1.2027	1.2039	1.2050	1.2062	56
1.2146	1.2158	1.2171	1.2183	1.2195	1.2208	55
1.2296	1.2309	1.2322	1.2335	1.2348	1.2361	54
1.2453	1.2467	1.2480	1.2494	1.2508	1.2521	53
1.2619	1.2633	1.2647	1.2661	1.2676	1.2690	52
1.2793	1.2807	1.2822	1.2837	1.2852	1.2867	51
1.2975	1.2991	1.3006	1.3022	1.3038	1.3054	50
1.3167	1.3184	1.3200	1.3217	1.3233	1.3250	49
1.3369	1.3386	1.3404	1.3421	1.3439	1.3456	48
1.3581	1.3600	1.3618	1.3636	1.3655	1.3673	47
1.3805	1.3824	1.3843	1.3863	1.3882	1.3902	46
1.4040	1.4056	1.4081	1.4101	1.4122	1.4142	45
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL COSECANT.

NATURAL SECANT.

Deg.	Minutes.						
	0	5	10	15	20	25	30
45	1.4142	1.4163	1.4183	1.4204	1.4225	1.4246	1.4267
46	1.4395	1.4417	1.4439	1.4461	1.4483	1.4505	1.4527
47	1.4663	1.4686	1.4709	1.4732	1.4755	1.4778	1.4802
48	1.4945	1.4969	1.4993	1.5018	1.5042	1.5067	1.5092
49	1.5242	1.5268	1.5294	1.5319	1.5345	1.5371	1.5398
50	1.5557	1.5584	1.5611	1.5639	1.5666	1.5694	1.5721
51	1.5890	1.5919	1.5947	1.5976	1.6005	1.6034	1.6064
52	1.6243	1.6273	1.6303	1.6334	1.6365	1.6396	1.6427
53	1.6616	1.6648	1.6681	1.6713	1.6746	1.6779	1.6812
54	1.7013	1.7047	1.7081	1.7116	1.7151	1.7185	1.7220
55	1.7434	1.7471	1.7507	1.7544	1.7581	1.7618	1.7655
56	1.7883	1.7921	1.7960	1.7999	1.8039	1.8078	1.8118
57	1.8361	1.8402	1.8443	1.8485	1.8527	1.8569	1.8611
58	1.8871	1.8915	1.8959	1.9004	1.9048	1.9093	1.9139
59	1.9416	1.9463	1.9510	1.9558	1.9606	1.9654	1.9703
60	2.0000	2.0050	2.0102	2.0152	2.0204	2.0256	2.0308
61	2.0627	2.0681	2.0735	2.0790	2.0846	2.0901	2.0957
62	2.1300	2.1359	2.1418	2.1477	2.1536	2.1596	2.1657
63	2.2027	2.2090	2.2153	2.2217	2.2282	2.2346	2.2411
64	2.2812	2.2880	2.2949	2.3018	2.3087	2.3158	2.3228
65	2.3662	2.3736	2.3811	2.3886	2.3961	2.4037	2.4114
66	2.4586	2.4666	2.4748	2.4829	2.4912	2.4995	2.5078
67	2.5593	2.5681	2.5770	2.5859	2.5949	2.6040	2.6131
68	2.6695	2.6791	2.6888	2.6986	2.7085	2.7185	2.7285
69	2.7904	2.8010	2.8117	2.8225	2.8334	2.8444	2.8554
70	2.9238	2.9355	2.9474	2.9593	2.9713	2.9835	2.9957
71	3.0715	3.0846	3.0977	3.1110	3.1244	3.1379	3.1515
72	3.2361	3.2506	3.2653	3.2801	3.2951	3.3102	3.3255
73	3.4203	3.4366	3.4532	3.4697	3.4867	3.5037	3.5209
74	3.6276	3.6464	3.6651	3.6840	3.7031	3.7224	3.7420
75	3.8637	3.8848	3.9061	3.9277	3.9495	3.9716	3.9939
76	4.1336	4.1578	4.1824	4.2072	4.2324	4.2579	4.2836
77	4.4454	4.4736	4.5021	4.5311	4.5604	4.5901	4.6202
78	4.8097	4.8429	4.8765	4.9106	4.9452	4.9802	5.0158
79	5.2408	5.2803	5.3205	5.3612	5.4026	5.4447	5.4874
80	5.7588	5.8067	5.8554	5.9049	5.9554	5.9963	6.0588
81	6.3924	6.4517	6.5121	6.5736	6.6363	6.7003	6.7655
82	7.1853	7.2604	7.3372	7.4156	7.4957	7.5776	7.6613
83	8.2055	8.3039	8.4046	8.5079	8.6138	8.7223	8.8337
84	9.5668	9.7010	9.8391	9.9812	10.1270	10.2780	10.4330
85	11.4740	11.6680	11.8680	12.0760	12.2910	12.5140	12.7450
86	14.3350	14.6400	14.9580	15.2900	15.6370	16.0000	16.3800
87	19.1070	19.6530	20.2300	20.8430	21.4940	22.1860	22.9250
88	28.6540	29.8990	31.2570	32.7450	34.3820	36.1910	38.2010
89	57.2990	62.5070	68.7570	76.3960	85.9460	98.2230	114.5900
Deg.	60	55	50	45	40	35	30
	Minutes.						

NATURAL COSECANT.

NATURAL SECANT.

Minutes.						Deg.
35	40	45	50	55	60	
1.4288	1.4310	1.4331	1.4352	1.4374	1.4395	44
1.4550	1.4572	1.4595	1.4617	1.4640	1.4663	43
1.4825	1.4849	1.4873	1.4897	1.4921	1.4945	42
1.5116	1.5141	1.5166	1.5192	1.5217	1.5242	41
1.5424	1.5450	1.5477	1.5503	1.5530	1.5557	40
1.5749	1.5777	1.5805	1.5833	1.5862	1.5890	39
1.6093	1.6123	1.6153	1.6182	1.6212	1.6243	38
1.6458	1.6489	1.6521	1.6552	1.6584	1.6616	37
1.6845	1.6878	1.6912	1.6945	1.6979	1.7013	36
1.7256	1.7291	1.7327	1.7362	1.7398	1.7434	35
1.7693	1.7730	1.7768	1.7806	1.7844	1.7883	34
1.8158	1.8198	1.8238	1.8279	1.8320	1.8361	33
1.8654	1.8697	1.8740	1.8783	1.8827	1.8871	32
1.9184	1.9230	1.9276	1.9322	1.9369	1.9416	31
1.9752	1.9801	1.9850	1.9900	1.9950	2.0000	30
2.0360	2.0413	2.0466	2.0519	2.0573	2.0627	29
2.1014	2.1070	2.1127	2.1185	2.1242	2.1300	28
2.1717	2.1778	2.1840	2.1902	2.1964	2.2027	27
2.2477	2.2543	2.2610	2.2676	2.2744	2.2812	26
2.3299	2.3371	2.3443	2.3515	2.3588	2.3662	25
2.4191	2.4269	2.4347	2.4426	2.4506	2.4586	24
2.5163	2.5247	2.5333	2.5419	2.5506	2.5593	23
2.6223	2.6316	2.6410	2.6504	2.6599	2.6695	22
2.7386	2.7488	2.7591	2.7694	2.7799	2.7904	21
2.8666	2.8778	2.8892	2.9006	2.9122	2.9238	20
3.0031	3.0206	3.0331	3.0458	3.0586	3.0715	19
3.1653	3.1792	3.1932	3.2074	3.2216	3.2361	18
3.3409	3.3565	3.3722	3.3881	3.4041	3.4203	17
3.5383	3.5559	3.5736	3.5915	3.6096	3.6279	16
3.7617	3.7816	3.8018	3.8222	3.8428	3.8637	15
4.0165	4.0394	4.0625	4.0859	4.1096	4.1336	14
4.3098	4.3362	4.3630	4.3901	4.4176	4.4454	13
4.6507	4.6817	4.7130	4.7448	4.7770	4.8097	12
5.0520	5.0886	5.1258	5.1636	5.2019	5.2408	11
5.5308	5.5749	5.6197	5.6653	5.7117	5.7588	10
6.1120	6.1661	6.2211	6.2772	6.3343	6.3924	9
6.8320	6.8998	6.9690	7.0396	7.1117	7.1853	8
7.7469	7.8344	7.9240	7.9971	8.1094	8.2055	7
8.9479	9.0651	9.1855	9.3092	9.4362	9.5668	6
10.5930	10.7580	10.9290	11.1040	11.2080	11.4740	5
12.9850	13.2350	13.4940	13.7630	14.0430	14.3350	4
16.7790	17.1980	17.6390	18.1030	18.5910	19.1070	3
23.7160	24.5620	25.4710	26.1500	27.5080	28.6540	2
39.9780	42.9760	45.8400	49.1140	52.8910	57.2990	1
137.5100	171.8900	229.1800	343.7700	687.5500	∞	0
25	20	15	10	5	0	Deg.
Minutes.						

NATURAL COSECANT.

CIRCUMFERENCE, AREA, AND CUBIC CONTENTS OF CIRCLES.

Diameter inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
$\frac{1}{16}$	0.1963	0.00307	0.03682	$1\frac{3}{4}$	4.4320	1.48489	17.8187	$2\frac{1}{8}$	8.4430	5.67266	68.0719
$\frac{1}{8}$	0.3927	0.01227	0.14726	$1\frac{7}{8}$	4.5160	1.52295	19.4754	$2\frac{3}{8}$	8.6394	5.93957	71.2749
$\frac{3}{16}$	0.5890	0.02761	0.33134	$1\frac{1}{2}$	4.7124	1.76715	21.2057	$2\frac{5}{8}$	8.8357	6.21262	74.5515
$\frac{1}{4}$	0.7854	0.04909	0.58905	$1\frac{1}{4}$	4.9087	1.91718	23.0097	$2\frac{7}{8}$	9.0321	6.49181	77.9017
$\frac{5}{16}$	0.9817	0.07670	0.92039	$1\frac{3}{8}$	5.1051	2.07394	24.8873	$2\frac{9}{8}$	9.2334	6.77713	81.3255
$\frac{3}{8}$	1.1781	0.11045	1.32536	$1\frac{1}{8}$	5.3014	2.23654	26.8385	3	9.4248	7.06858	84.8230
$\frac{7}{16}$	1.3744	0.15033	1.80396	$1\frac{1}{4}$	5.4978	2.40328	28.8634	$3\frac{1}{8}$	9.6211	7.36618	88.3941
$\frac{1}{2}$	1.5708	0.19635	2.35619	$1\frac{3}{8}$	5.6941	2.58015	30.9619	$3\frac{3}{8}$	9.8175	7.66990	92.0338
$\frac{9}{16}$	1.7671	0.24850	2.98206	$1\frac{1}{2}$	5.8905	2.76116	33.1340	$3\frac{5}{8}$	10.0138	7.97977	95.7572
$\frac{5}{8}$	1.9635	0.30680	3.68155	$1\frac{5}{8}$	6.0868	2.94831	35.3797	$3\frac{7}{8}$	10.2102	8.29577	99.5492
$\frac{3}{4}$	2.1598	0.37122	4.45468	2	6.2832	3.14159	37.6991	$3\frac{9}{8}$	10.4065	8.61790	103.4148
$\frac{7}{8}$	2.3562	0.44179	5.30143	$2\frac{1}{8}$	6.4795	3.34101	40.0921	$3\frac{11}{8}$	10.6029	8.94618	107.3541
1	2.5525	0.51849	6.22182	$2\frac{3}{8}$	6.6759	3.54556	42.5588	$3\frac{1}{2}$	10.7992	9.28058	111.3670
$1\frac{1}{16}$	2.7489	0.60132	7.21584	$2\frac{5}{8}$	6.8722	3.75825	45.0990	$3\frac{3}{4}$	10.9956	9.62113	115.4535
$1\frac{1}{8}$	2.9452	0.69029	8.28349	$2\frac{7}{8}$	7.0686	3.97608	47.7129	$3\frac{5}{8}$	11.1919	9.96781	119.6137
$1\frac{1}{4}$	3.1416	0.78540	9.42477	$2\frac{9}{8}$	7.2649	4.20004	50.4005	$3\frac{7}{8}$	11.3883	10.32062	123.8475
$1\frac{3}{8}$	3.3379	0.88664	10.63970	$2\frac{11}{8}$	7.4613	4.43014	53.1616	$3\frac{9}{8}$	11.5846	10.67957	128.1549
$1\frac{1}{2}$	3.5343	0.99402	11.92830	$2\frac{3}{4}$	7.6576	4.66637	55.9964	$3\frac{11}{8}$	11.7810	11.04466	132.5359
$1\frac{5}{8}$	3.7306	1.10753	13.29040	$2\frac{5}{4}$	7.8510	4.90874	58.9049	$3\frac{1}{2}$	11.9773	11.41588	136.9906
$1\frac{3}{4}$	3.9270	1.22718	14.72620	$2\frac{7}{4}$	8.0503	5.15724	61.8869	$3\frac{3}{4}$	12.1737	11.79324	141.5189
$1\frac{7}{8}$	4.1233	1.35297	16.23560	$2\frac{9}{4}$	8.2467	5.41188	64.9426	$3\frac{5}{4}$	12.3700	12.17674	146.1209

Diameter. inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter. inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter. inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
4	12.5664	12.56637	150.7964	5 $\frac{3}{16}$	16.8861	22.69064	272.2877	7 $\frac{1}{2}$	23.5619	44.17865	530.1438
4 $\frac{1}{16}$	12.7627	12.96214	155.5457	5 $\frac{7}{16}$	17.0824	23.22140	278.6568	7 $\frac{3}{16}$	23.9546	45.66354	547.9625
4 $\frac{1}{8}$	12.9591	13.36404	160.3685	5 $\frac{1}{2}$	17.2788	23.75829	285.0995	7 $\frac{5}{16}$	24.3473	47.17298	566.0757
4 $\frac{3}{16}$	13.1554	13.77208	165.2650	5 $\frac{9}{16}$	17.4751	24.30132	291.6159	7 $\frac{7}{16}$	24.7400	48.70696	584.4835
4 $\frac{1}{2}$	13.3518	14.18625	170.2351	5 $\frac{11}{16}$	17.6715	24.85049	298.2059	8	25.1327	50.26548	603.1858
4 $\frac{5}{16}$	13.5481	14.60656	175.2788	5 $\frac{13}{16}$	17.8678	25.40579	304.8695	8 $\frac{1}{16}$	25.5254	51.84855	622.1826
4 $\frac{3}{8}$	13.7445	15.03301	180.3961	5 $\frac{1}{2}$	18.0642	25.96723	311.6068	8 $\frac{1}{8}$	25.9181	53.45616	641.4739
4 $\frac{7}{16}$	13.9408	15.46559	185.5871	5 $\frac{3}{8}$	18.2605	26.53480	318.4176	8 $\frac{3}{16}$	26.3108	55.08832	661.0598
4 $\frac{1}{2}$	14.1372	15.90431	190.8518	5 $\frac{1}{2}$	18.4569	27.10851	325.3021	8 $\frac{1}{2}$	26.7035	56.74502	680.9402
4 $\frac{9}{16}$	14.3335	16.34917	196.1901	5 $\frac{1}{2}$	18.6532	27.68835	332.2602	8 $\frac{5}{16}$	27.0962	58.42626	701.1151
4 $\frac{5}{8}$	14.5299	16.80016	201.6019	6	18.8496	28.27433	339.2920	8 $\frac{3}{8}$	27.4889	60.13205	721.5846
4 $\frac{11}{16}$	14.7262	17.25728	207.0874	6 $\frac{1}{16}$	19.2423	29.46470	353.5764	8 $\frac{7}{16}$	27.8816	61.86238	742.3485
4 $\frac{3}{4}$	14.9226	17.72055	212.6465	6 $\frac{1}{8}$	19.6350	30.67962	368.1554	9	28.2743	63.61725	763.4070
4 $\frac{13}{16}$	15.1189	18.18994	218.2793	6 $\frac{3}{16}$	20.0277	31.91907	383.0289	9 $\frac{1}{16}$	28.6670	65.39667	784.7600
4 $\frac{3}{4}$	15.3153	18.66548	223.9853	6 $\frac{1}{2}$	20.4204	33.18307	398.1969	9 $\frac{1}{8}$	29.0597	67.20063	806.4076
4 $\frac{15}{16}$	15.5116	19.14715	229.7658	6 $\frac{5}{16}$	20.8131	34.47162	413.6594	9 $\frac{3}{16}$	29.4524	69.02914	828.3496
5	15.7080	19.63495	235.6195	6 $\frac{3}{8}$	21.2058	35.78470	429.4164	9 $\frac{1}{2}$	29.8451	70.88248	850.5862
5 $\frac{1}{16}$	15.9043	20.12890	241.5468	6 $\frac{7}{16}$	21.5984	37.12233	445.4680	9 $\frac{5}{16}$	30.2378	72.75978	873.1173
5 $\frac{1}{8}$	16.1007	20.62897	247.5477	7	21.9911	38.48451	461.8141	9 $\frac{3}{8}$	30.6305	74.66791	895.9430
5 $\frac{3}{16}$	16.2970	21.13519	253.6223	7 $\frac{1}{16}$	22.3838	39.87123	478.4547	9 $\frac{7}{16}$	31.0232	76.58859	919.0631
5 $\frac{1}{2}$	16.4934	21.64754	259.7705	7 $\frac{1}{8}$	22.7765	41.28249	495.3895	10	31.4159	78.53982	942.4779
5 $\frac{5}{16}$	16.6897	22.16602	265.9923	7 $\frac{3}{16}$	23.1692	42.71830	512.6196	10 $\frac{1}{8}$	31.8086	80.51558	966.1870

220 CIRCUMFERENCE, AREA, AND CUBIC CONTENTS OF CIRCLES.

Diameter in inches.	Circum- ference inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
10 1	32.2013	82.51589	990.1907	13	40.8407	132.73229	1592.7875	15 1	49.4801	194.82783	2337.9340
10 2	32.5940	84.54075	1014.4890	13 1	41.2334	135.29711	1623.5653	15 1	49.8728	197.93261	2375.1913
10 3	32.9867	86.59015	1039.0818	13 1	41.6261	137.88647	1654.5176	16	50.2655	201.06193	2416.3432
10 4	33.3794	88.66409	1063.9691	13 1	42.0188	140.50037	1686.0044	16 1	50.6582	204.21579	2450.5895
10 5	33.7721	90.76257	1089.1509	13 1	42.4115	143.13882	1717.5458	16 1	51.0509	207.39420	2488.7304
10 6	34.1648	92.88560	1114.6272	13 1	42.8042	145.80181	1749.6217	16 1	51.4436	210.59715	2527.1658
11	34.5575	95.03318	1140.3981	13 1	43.1969	148.48934	1781.8721	16 1	51.8363	213.82465	2565.8958
11 1	34.9502	97.20529	1166.4635	13 1	43.5896	151.20142	1814.4170	16 1	52.2290	217.07669	2604.9203
11 2	35.3429	99.40195	1192.8235	14	43.9823	153.93801	1847.2565	16 1	52.6217	220.35327	2644.2393
11 3	35.7356	101.62316	1219.4779	14 1	44.3750	156.69921	1880.3905	16 1	53.0144	223.65440	2683.8528
11 4	36.1283	103.86891	1246.4269	14 1	44.7677	159.48491	1913.8190	17	53.4071	226.98007	2723.7608
11 5	36.5210	106.13920	1273.6763	14 1	45.1604	162.29517	1947.5420	17 1	53.7998	230.33028	2763.9634
11 6	36.9137	108.43403	1301.2084	14 1	45.5531	165.12996	1981.5596	17 1	54.1925	233.70504	2804.4605
12	37.3064	110.75341	1329.0410	14 1	45.9458	167.9930	2015.8716	17 1	54.5852	237.10434	2845.2521
12 1	37.6991	113.09734	1357.1680	14 1	46.3385	170.87319	2050.4783	17 1	54.9779	240.52819	2886.3382
12 2	38.0918	115.46580	1385.5896	14 1	46.7312	173.78162	2085.3794	17 1	55.3706	243.97658	2927.7189
12 3	38.4845	117.85881	1414.3057	15	47.1239	176.71459	2120.5750	17 1	55.7633	247.44951	2969.3941
12 4	38.8772	120.27637	1443.1964	15 1	47.5166	179.67210	2156.0652	17 1	56.1560	250.94698	3011.3638
12 5	39.2699	122.71846	1472.6216	15 1	47.9093	182.65416	2191.8499	18	56.5487	254.46901	3053.8281
12 6	39.6626	125.18510	1502.2213	15 1	48.3020	185.66076	2227.9291	18 1	56.9414	258.01557	3096.1868
13	40.0480	127.67629	1531.9955	15 1	48.6947	188.69191	2264.3029	18 1	57.3341	261.58668	3139.0401
13 1	40.4480	130.19202	1562.3042	15 1	49.0874	191.74760	2300.9712	18 1	57.7268	265.18233	3182.1879

Diameter. in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
18 $\frac{1}{8}$	58.1195	268.80252	3225.6303	21 $\frac{1}{4}$	66.7588	354.65636	4255.8763	24	75.3982	452.38934	5428.6721
18 $\frac{1}{4}$	58.5122	272.44726	3269.3671	21 $\frac{1}{2}$	67.1515	358.84106	4306.0927	24 $\frac{1}{2}$	76.1836	461.86320	5542.3585
18 $\frac{3}{8}$	58.9049	276.11654	3313.3985	21 $\frac{3}{8}$	67.5442	363.05030	4356.6036	24 $\frac{3}{8}$	76.9690	471.43520	5657.2230
18 $\frac{1}{2}$	59.2976	279.81037	3357.7244	21 $\frac{1}{2}$	67.9369	367.28409	4407.4091	24 $\frac{1}{2}$	77.7544	481.10550	5773.2655
19	59.6903	283.52874	3402.3449	21 $\frac{1}{2}$	68.3296	371.54242	4458.5090	25	78.5398	490.87390	5890.4862
19 $\frac{1}{8}$	60.0830	287.27165	3447.2598	21 $\frac{3}{8}$	68.7223	375.82529	4509.9085	25 $\frac{1}{8}$	79.3252	500.74840	6008.8850
19 $\frac{1}{4}$	60.4757	291.03911	3492.4693	22	69.1150	380.13271	4561.5926	25 $\frac{1}{4}$	80.1106	510.70520	6128.4619
19 $\frac{3}{8}$	60.8684	294.83111	3537.9733	22 $\frac{1}{8}$	69.5077	384.46467	4613.5761	25 $\frac{3}{8}$	80.8960	520.76810	6249.2168
19 $\frac{1}{2}$	61.2611	298.64765	3583.7718	22 $\frac{1}{4}$	69.9004	388.82118	4665.8542	26	81.6814	530.92920	6371.1499
19 $\frac{3}{4}$	61.6538	302.48874	3629.8649	22 $\frac{3}{8}$	70.2931	393.20223	4718.4268	26 $\frac{1}{8}$	82.4668	541.18840	6494.2611
19 $\frac{1}{2}$	62.0465	306.35437	3676.2525	22 $\frac{1}{2}$	70.6858	397.60782	4771.2939	26 $\frac{1}{2}$	83.2522	551.54590	6618.5503
19 $\frac{3}{4}$	62.4392	310.24455	3722.9346	22 $\frac{3}{4}$	71.0785	402.03796	4824.4555	26 $\frac{3}{4}$	84.0376	562.00150	6744.0177
20	62.8319	314.15927	3769.9112	23	71.4712	406.49264	4877.9117	27	84.8230	572.55530	6870.6631
20 $\frac{1}{8}$	63.2246	318.09853	3817.1823	23 $\frac{1}{8}$	71.8639	410.97186	4931.6624	27 $\frac{1}{8}$	85.6034	583.20720	6998.4867
20 $\frac{1}{4}$	63.6173	322.06233	3864.7480	23 $\frac{1}{4}$	72.2566	415.47563	4985.7076	27 $\frac{1}{4}$	86.3938	593.95740	7127.4883
20 $\frac{3}{8}$	64.0100	326.05068	3912.6082	23 $\frac{3}{8}$	72.6493	420.00394	5040.0473	27 $\frac{3}{8}$	87.1792	604.80570	7257.6681
20 $\frac{1}{2}$	64.4026	330.06358	3960.7629	23 $\frac{1}{2}$	73.0420	424.55680	5094.6816	28	87.9646	615.75220	7389.0259
20 $\frac{3}{4}$	64.7953	334.10102	4009.2122	23 $\frac{3}{4}$	73.4347	429.13420	5149.6104	28 $\frac{1}{4}$	88.7500	626.79680	7521.5618
20 $\frac{1}{2}$	65.1880	338.16300	4057.9560	24	73.8274	433.73614	5204.8337	28 $\frac{1}{2}$	89.5354	637.93970	7655.2759
20 $\frac{3}{4}$	65.5807	342.24952	4106.9943	24 $\frac{1}{4}$	74.2201	438.36232	5260.3515	28 $\frac{3}{4}$	90.3208	649.18070	7790.1680
21	65.9734	346.36059	4156.3271	24 $\frac{1}{2}$	74.6128	443.01365	5316.1638	29	91.1062	660.51990	7926.2383
21 $\frac{1}{8}$	66.3661	350.49620	4205.9544	24 $\frac{3}{8}$	75.0055	447.68923	5372.2707	29 $\frac{1}{8}$	91.8916	671.95720	8063.4866

Diameter. in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
29 $\frac{1}{2}$	92.6770	683.4928	8201.9130	35	109.9557	962.1127	11545.3530	40 $\frac{1}{2}$	127.2345	1288.2493	15458.9920
29 $\frac{3}{4}$	93.4624	695.1265	8341.5175	35 $\frac{1}{2}$	110.7411	975.9063	11710.8766	40 $\frac{3}{4}$	128.0199	1304.2027	15650.4328
30	94.2478	706.8583	8482.3002	35 $\frac{3}{4}$	111.6265	989.7980	11877.5764	41	128.8055	1320.2543	15843.0517
30 $\frac{1}{4}$	95.0332	718.6884	8624.2609	35 $\frac{1}{2}$	112.5119	1003.7879	12045.4553	41 $\frac{1}{4}$	129.5907	1336.4041	16036.8487
30 $\frac{1}{2}$	95.8186	730.6166	8767.3997	36	113.0973	1017.8760	12214.5122	41 $\frac{1}{2}$	130.3761	1352.6520	16231.8238
30 $\frac{3}{4}$	96.6040	742.6431	8911.7166	36 $\frac{1}{2}$	113.8827	1032.0623	12384.7473	41 $\frac{3}{4}$	131.1615	1368.9981	16427.9770
31	97.3894	754.7676	9057.2116	36 $\frac{3}{4}$	114.6681	1046.3467	12556.1604	42	131.9469	1385.4424	16625.3083
31 $\frac{1}{4}$	98.1748	766.9904	9203.8847	36 $\frac{1}{2}$	115.4535	1060.7293	12728.7518	42 $\frac{1}{4}$	132.7323	1401.9848	16823.8177
31 $\frac{1}{2}$	98.9602	779.3113	9351.7359	37	116.2389	1075.2101	12902.5210	42 $\frac{1}{2}$	133.5177	1418.6254	17023.5051
31 $\frac{3}{4}$	99.7456	791.7304	9500.7652	37 $\frac{1}{2}$	117.0243	1089.7890	13077.4684	42 $\frac{3}{4}$	134.3031	1435.3642	17224.3707
32	100.5310	804.2477	9650.9726	37 $\frac{3}{4}$	117.8097	1104.4662	13253.5940	43	135.0885	1452.2012	17426.4144
32 $\frac{1}{4}$	101.3164	816.8632	9802.3581	37 $\frac{1}{2}$	118.5951	1119.2415	13430.8976	43 $\frac{1}{4}$	135.8739	1469.1363	17629.6362
32 $\frac{1}{2}$	102.1018	829.5768	9954.9217	38	119.3805	1134.1149	13609.3793	43 $\frac{1}{2}$	136.6593	1486.1697	17834.0361
32 $\frac{3}{4}$	102.8872	842.3886	10108.6634	38 $\frac{1}{2}$	120.1659	1149.0864	13789.0392	43 $\frac{3}{4}$	137.4447	1503.3012	18039.6140
33	103.6726	855.2986	10263.5832	38 $\frac{3}{4}$	120.9513	1164.1564	13969.8771	44	138.2301	1520.5308	18246.3701
33 $\frac{1}{4}$	104.4580	868.3068	10419.6811	38 $\frac{1}{2}$	121.7367	1179.3244	14151.8931	44 $\frac{1}{4}$	139.0155	1537.8587	18454.3042
33 $\frac{1}{2}$	105.2434	881.4131	10576.9571	39	122.5221	1194.5906	14335.0872	44 $\frac{1}{2}$	139.8009	1555.2847	18663.4165
33 $\frac{3}{4}$	106.0288	894.6176	10735.4111	39 $\frac{1}{2}$	123.3075	1209.9550	14519.4595	44 $\frac{3}{4}$	140.5863	1572.8089	18873.7069
34	106.8142	907.9203	10895.0433	39 $\frac{3}{4}$	124.0929	1225.4175	14705.0098	45	141.3717	1590.4313	19085.1753
34 $\frac{1}{4}$	107.5994	921.3211	11055.8536	39 $\frac{1}{2}$	124.8783	1240.9782	14891.7382	45 $\frac{1}{4}$	142.1571	1608.1518	19297.8219
34 $\frac{1}{2}$	108.3849	934.8202	11217.8420	40	125.6637	1256.6371	15079.6447	45 $\frac{1}{2}$	142.9425	1625.9705	19511.6465
34 $\frac{3}{4}$	109.1703	948.4174	11381.0084	40 $\frac{1}{2}$	126.4491	1272.3941	15268.7293	45 $\frac{3}{4}$	143.7279	1643.8874	19726.6493

Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.	Diameter in inches.	Circum- ference in inches.	Area in square inches.	Contents of one foot in length in cubic inches.
46	144.5133	1661.9025	19942.8301	50 $\frac{3}{4}$	159.4358	2022.8421	24274.1046	55 $\frac{1}{4}$	174.3684	2419.2227	29030.6722
46 $\frac{1}{4}$	145.2987	1680.0158	20160.1891	51	160.2212	2042.8206	24513.8474	55 $\frac{1}{2}$	175.1438	2441.0666	29292.7989
46 $\frac{1}{2}$	146.0841	1698.2272	20378.7261	51 $\frac{1}{4}$	161.0066	2062.8974	24755.8485	56	175.9292	2463.0086	29556.1036
46 $\frac{3}{4}$	146.8695	1716.5316	20598.4412	51 $\frac{1}{2}$	161.7920	2083.0723	24996.8675	56 $\frac{1}{4}$	176.7146	2485.0489	29820.5865
47	147.6549	1734.9445	20819.3345	51 $\frac{3}{4}$	162.5774	2103.3454	25240.1443	56 $\frac{1}{2}$	177.5100	2507.1873	30086.2474
47 $\frac{1}{4}$	148.4403	1753.4505	21041.4058	52	163.3628	2123.7166	25484.5995	56 $\frac{3}{4}$	178.2954	2529.4239	30353.0865
47 $\frac{1}{2}$	149.2257	1772.0546	21264.6552	52 $\frac{1}{4}$	164.1482	2144.1861	25730.2328	57	179.0808	2551.7586	30621.1036
47 $\frac{3}{4}$	150.0110	1790.7569	21489.0827	52 $\frac{1}{2}$	164.9336	2164.7537	25977.0442	57 $\frac{1}{4}$	179.8662	2574.1916	30890.2988
48	150.7964	1809.5574	21714.6884	52 $\frac{3}{4}$	165.7190	2185.4195	26225.0336	57 $\frac{1}{2}$	180.6516	2596.7227	31160.6721
48 $\frac{1}{4}$	151.5818	1828.4560	21941.4721	53	166.5044	2206.1834	26474.2012	57 $\frac{3}{4}$	181.4370	2619.3520	31432.2235
48 $\frac{1}{2}$	152.3672	1847.4528	22169.4339	53 $\frac{1}{4}$	167.2898	2227.0456	26724.5469	58	182.2224	2642.0794	31704.9531
48 $\frac{3}{4}$	153.1526	1866.5478	22398.5738	53 $\frac{1}{2}$	168.0752	2248.0059	26976.6706	58 $\frac{1}{4}$	183.0078	2664.9051	31978.8607
49	153.9380	1885.7410	22628.8918	53 $\frac{3}{4}$	168.8606	2269.0644	27228.7725	58 $\frac{1}{2}$	183.7932	2687.8289	32253.9464
49 $\frac{1}{4}$	154.7234	1905.0323	22860.3879	54	169.6460	2290.0210	27482.6524	58 $\frac{3}{4}$	184.5786	2710.8508	32530.2102
49 $\frac{1}{2}$	155.5088	1924.4218	23093.0621	54 $\frac{1}{4}$	170.4314	2311.4759	27737.7105	59	185.3640	2733.9710	32807.6521
49 $\frac{3}{4}$	156.2942	1943.9095	23326.9144	54 $\frac{1}{2}$	171.2168	2332.8289	27993.9467	59 $\frac{1}{4}$	186.1494	2757.1893	33086.2721
50	157.0796	1963.4954	23561.9448	54 $\frac{3}{4}$	172.0022	2354.2801	28251.3609	59 $\frac{1}{2}$	186.9348	2780.5058	33366.0702
50 $\frac{1}{4}$	157.8650	1983.1794	23798.1533	55	172.7876	2375.8294	28509.9532	59 $\frac{3}{4}$	187.7202	2803.9205	33647.0464
50 $\frac{1}{2}$	158.6504	2002.9617	24035.5399	55 $\frac{1}{4}$	173.5730	2397.4770	28769.7237	60	188.5056	2827.4334	33929.2000

SPECIFIC GRAVITIES OF MATERIALS.

	Weight of a cubic foot in lbs. avoirdupois.
GASES at 32° Fahr., and under the pressure of one atmosphere of 2116.4 lbs. on the square foot:	
Air	0.080728
Carbonic acid.....	0.12344
Hydrogen	0.005592
Oxygen	0.089256
Nitrogen.....	0.078596
Steam (ideal).....	0.05022
Æther vapor (ideal).....	0.2093
Bisulphuret-of-carbon vapour (ideal).....	0.2137
Olefiant gas.....	0.0795

	Weight of a cubic foot in lbs. avoirdupois.	Specific gravity, pure water = 1.
LIQUIDS at 32° Fahr. (except water, which is taken at 39° 4 Fahr.):		
Water, pure, at 39° 4.....	62.425	1.000
“ sea, ordinary	64.05	1.026
Alcohol, pure.....	49.38	0.791
“ proof spirit.....	57.18	0.916
Æther.....	44.70	0.716
Mercury	848.75	13.596
Naphtha.....	52.94	0.848
Oil, linseed.....	58.68	0.940
“ olive.....	57.12	0.915
“ whale.....	57.62	0.923
“ of turpentine.....	54.31	0.870
Petroleum.....	54.81	0.878
SOLID MINERAL SUBSTANCES, non- metallic:		
Basalt.....	187.3	3.00
Brick.....	125 to 135	2 to 2.167
Brickwork.....	112	1.8
Chalk.....	117 to 174	1.87 to 2.78
Clay	120	1.92
Coal, anthracite.....	100	1.602
“ bituminous.....	77.4 to 89.9	1.24 to 1.44
Coke.....	62.43 to 103.6	1.00 to 1.66
Felspar	162.3	2.6
Flint.....	164.2	2.63

	Weight of a cubic foot in lbs. avoirdupois.	Specific gravity, pure water = 1.
SOLID MINERAL SUBSTANCES—con-		
tinued:		
Glass, crown, average	156	2.5
“ flint.....	187	3.0
“ green.....	169	2.7
“ plate.....	169	2.7
Granite.....	164 to 172	2.63 to 2.76
Gypsum.....	143.6	2.3
Limestone, (including marble)...	169 to 175	2.7 to 2.8
“ magnesian.....	178	2.86
Marl.....	100 to 119	1.6 to 1.9
Masonry.....	116 to 144	1.85 to 2.3
Mortar.....	109	1.75
Mud.....	102	1.63
Quartz.....	165	2.65
Sand (damp).....	118	1.9
“ (dry).....	88.6	1.42
Sandstone, average.....	144	2.3
“ various kinds.....	130 to 157	2.08 to 2.52
Shale.....	162	2.6
Slate.....	175 to 181	2.8 to 2.9
Trap.....	170	2.72
METALS, solid:		
Brass, cast.....	487 to 524.4	7.8 to 8.4
“ wire.....	533	8.54
Bronze.....	524	8.4
Copper, cast.....	537	8.6
“ sheet.....	549	8.8
“ hammered.....	556	8.9
Gold.....	1186 to 1224	19 to 19.6
Iron, cast, various.....	434 to 456	6.95 to 7.3
“ average.....	444	7.11
Iron, wrought, various.....	474 to 487	7.6 to 7.8
“ average.....	480	7.69
Lead.....	712	11.4
Platinum.....	1311 to 1373	21 to 22
Silver.....	655	10.5
Steel.....	487 to 493	7.8 to 7.9
Tin.....	456 to 468	7.3 to 7.5
Zinc.....	424 to 449	6.8 to 7.2
TIMBER: *		
Ash.....	47	0.753
Bamboo.....	25	0.4
Beech.....	43	0.69

	Weight of a cubic foot in lbs. avoirdupois.	Specific gravity, pure water = 1.
TIMBER: *—continued.		
Birch.....	44.4	0.711
Blue-gum.....	52.5	0.843
Box.....	60	0.96
Bullet-tree.....	65.3	1.046
Cabacalli.....	56.2	0.9
Cedar of Lebanon.....	30.4	0.486
Chestnut.....	33.4	0.535
Cowrie.....	36.2	0.579
Ebony, West Indian.....	74.5	1.193
Elm.....	34	0.544
Fir, red pine.....	30 to 44	0.48 to 0.7
“ spruce.....	30 to 44	0.48 to 0.7
“ American yellow pine.....	29	0.46
“ larch.....	31 to 35	0.5 to 0.56
Greenhart.....	62.5	1.001
Hawthorn.....	57	0.91
Hazel.....	54	0.86
Holly.....	47	0.76
Hornbeam.....	47	0.76
Laburnum.....	57	0.92
Lancewood.....	42 to 63	0.675 to 1.01
Larch. (See “fir”.)		
Lignum-vitæ.....	41 to 83	0.65 to 1.33
Locust.....	44	0.71
Mahogany, Honduras.....	35	0.56
“ Spanish.....	53	0.85
Maple.....	49	0.79
Mora.....	57	0.92
Oak, European.....	43 to 62	0.69 to 0.99
“ American red.....	54	0.87
Poon.....	36	0.58
Saul.....	60	0.96
Sycamore.....	37	0.59
Teak, Indian.....	41 to 55	0.66 to 0.88
“ African.....	61	0.98
Tonka.....	62 to 66	0.99 to 1.06
Water-gum.....	62.5	1.001
Willow.....	25	0.4
Yew.....	50	0.8

*The timber in every case is supposed to be dry.

WEIGHT OF A SUPERFICIAL INCH OF WROUGHT AND CAST IRON.

(From one-sixteenth to one-inch thickness.)

Thickness in inches.	WROUGHT IRON. Cubic foot = 480 lbs.	CAST IRON. Cubic foot = 450 lbs.
	Weight in lbs.	Weight in lbs.
$\frac{1}{16}$	0.017356	0.0163
$\frac{1}{8}$	0.0347	0.0326
$\frac{3}{16}$	0.0520	0.0489
$\frac{1}{4}$	0.0694	0.0652
$\frac{5}{16}$	0.0867	0.0815
$\frac{3}{8}$	0.1041	0.0978
$\frac{7}{16}$	0.1214	0.1141
$\frac{1}{2}$	0.1388	0.1304
$\frac{9}{16}$	0.1562	0.1467
$\frac{5}{8}$	0.1735	0.1630
$\frac{11}{16}$	0.1909	0.1793
$\frac{3}{4}$	0.2082	0.1956
$\frac{13}{16}$	0.2256	0.2119
$\frac{7}{8}$	0.2429	0.2282
$\frac{15}{16}$	0.2603	0.2445
1	0.2777	0.2608

WEIGHT PER SQUARE FOOT IN POUNDS AVOIRDUPOIS.

Thickness in inches.	Wrought Iron.	Cast Iron.	Copper, sheet.	Lead.	Zinc.
	480 lbs. per cubic foot.	450 lbs. per cubic foot.	549 lbs. per cubic foot.	712 lbs. per cubic foot.	436 lbs. per cubic foot.
$\frac{1}{16}$	2.50	2.34	2.86	3.71	2.27
$\frac{1}{8}$	5.00	4.69	5.72	7.42	4.54
$\frac{3}{16}$	7.50	7.03	8.58	11.12	6.81
$\frac{1}{4}$	10.00	9.37	11.44	14.83	9.08
$\frac{5}{16}$	12.50	11.72	14.30	18.54	11.35
$\frac{3}{8}$	15.00	14.06	17.16	22.25	13.62
$\frac{7}{16}$	17.50	16.41	20.02	25.96	15.89
$\frac{1}{2}$	20.00	18.75	22.88	29.66	18.16
$\frac{9}{16}$	22.50	21.09	25.74	33.37	20.43
$\frac{5}{8}$	25.00	23.44	28.60	37.10	22.70
$\frac{11}{16}$	27.50	25.78	31.46	40.79	24.97
$\frac{3}{4}$	30.00	28.12	34.32	44.50	27.24
$\frac{13}{16}$	32.50	30.47	37.18	48.20	29.51
$\frac{7}{8}$	35.00	32.81	40.04	51.91	31.78
$\frac{15}{16}$	37.50	35.16	42.90	55.62	34.05
1	40.00	37.50	45.75	59.33	36.33

WEIGHT OF A LINEAL FOOT OF FLAT AND SQUARE
BAR IRON IN POUNDS AVOIRDUPOIS.

(480 pounds per cubic foot.)

Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.
1	1	0.104	1 1/2	1	5.000	2 1/4	1	1.875
1 1/4	1 1/4	0.208	"	1 1/8	5.625	"	1 1/8	2.813
1 1/2	1 1/2	0.208	"	1 1/4	6.250	"	1 1/4	3.750
"	1 3/4	0.416	"	1 3/8	6.874	"	1 3/8	4.687
"	2	0.832	"	1 1/2	7.500	"	1 1/2	5.624
3/4	2 1/4	0.312	1 3/4	1 5/8	0.739	"	1 5/8	6.562
"	"	0.624	"	1 3/4	1.459	"	1 3/4	7.500
"	"	0.937	"	1 3/8	2.187	"	1 3/8	8.437
"	"	1.249	"	1 3/4	2.916	"	1 3/4	9.374
"	"	1.562	"	1 3/8	3.646	"	1 3/8	10.310
"	"	1.874	"	1 3/4	4.375	"	1 3/4	11.250
1	1 1/8	0.416	"	1 3/4	5.103	"	1 3/4	12.190
"	"	0.833	"	1	5.833	"	1 3/4	13.120
"	"	1.249	"	1 1/8	6.562	"	1 7/8	14.060
"	"	1.667	"	1 1/4	7.291	"	2	15.000
"	"	2.089	"	1 3/8	8.020	"	2 1/8	15.940
"	"	2.500	"	1 1/2	8.750	"	2 1/4	17.810
"	"	2.916	"	1 3/4	9.478	2 1/2	2 1/4	1.041
"	1	3.333	"	1 3/8	10.930	"	1 3/8	2.089
1 1/4	1 1/8	0.521	2	1 3/4	0.833	"	1 3/8	3.125
"	1 1/4	1.041	"	1 1/4	1.667	"	1 3/8	4.166
"	"	1.562	"	1 3/8	2.500	"	1 3/8	5.208
"	"	2.089	"	1 1/2	3.333	"	1 3/8	6.250
"	"	2.603	"	1 3/4	4.166	"	1 3/8	7.291
"	"	3.124	"	1 3/4	5.000	"	1	8.333
"	"	3.646	"	1 3/8	5.833	"	1 1/4	9.398
"	1	4.166	"	1	6.666	"	1 1/4	10.410
"	1 1/8	4.687	"	1 1/8	7.500	"	1 1/4	11.460
"	1 1/4	5.728	"	1 1/4	8.333	"	1 1/4	12.500
1 1/2	1 1/4	0.624	"	1 3/8	9.156	"	1 1/4	13.540
"	1 3/4	1.250	"	1 1/2	10.000	"	1 3/4	14.580
"	"	1.875	"	1 3/8	10.830	"	1 3/4	15.620
"	"	2.500	"	1 3/4	11.660	"	2	16.660
"	"	3.125	"	1 7/8	12.500	"	2 1/4	17.710
"	"	3.750	"	2	13.330	"	2 1/4	18.750
"	"	4.375	2 1/4	1 3/8	0.937	"	2 1/2	20.820

Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.
2 $\frac{1}{2}$	2 $\frac{3}{8}$	19.800	3 $\frac{1}{4}$	2	21.660	4	2	26.660
2 $\frac{3}{4}$	1 $\frac{1}{2}$	1.146	"	2 $\frac{1}{4}$	24.370	"	2 $\frac{1}{4}$	30.000
"	1 $\frac{3}{8}$	2.292	"	2 $\frac{1}{2}$	27.080	"	2 $\frac{1}{2}$	33.330
"	1 $\frac{1}{4}$	3.437	"	2 $\frac{3}{4}$	29.790	"	2 $\frac{3}{4}$	36.660
"	1 $\frac{1}{8}$	4.583	"	3	32.500	"	3	40.000
"	5 $\frac{1}{8}$	5.729	"	3 $\frac{1}{4}$	24.200	"	3 $\frac{1}{4}$	43.330
"	5 $\frac{1}{4}$	6.874	3 $\frac{1}{2}$	1 $\frac{1}{4}$	2.916	"	3 $\frac{1}{2}$	46.660
"	5	8.020	"	1 $\frac{1}{2}$	5.833	"	3 $\frac{3}{4}$	50.000
"	4 $\frac{7}{8}$	9.154	"	1 $\frac{3}{4}$	8.750	"	4	53.330
"	1	10.310	"	1	11.660	4 $\frac{1}{4}$	1 $\frac{1}{4}$	3.541
"	1 $\frac{1}{8}$	11.460	"	1 $\frac{1}{4}$	14.580	"	1 $\frac{1}{2}$	7.082
"	1 $\frac{1}{4}$	12.600	"	1 $\frac{1}{2}$	17.500	"	1 $\frac{3}{4}$	10.620
"	1 $\frac{3}{8}$	13.750	"	1 $\frac{3}{4}$	20.430	"	1	14.160
"	1 $\frac{1}{2}$	14.900	"	2	23.330	"	1 $\frac{1}{4}$	16.800
"	1 $\frac{3}{4}$	16.030	"	2 $\frac{1}{4}$	26.250	"	1 $\frac{1}{2}$	21.330
"	1 $\frac{7}{8}$	17.190	"	2 $\frac{1}{2}$	29.160	"	1 $\frac{3}{4}$	24.780
"	2	18.330	"	2 $\frac{3}{4}$	32.080	"	2	28.330
"	2 $\frac{1}{8}$	19.480	"	3	35.000	"	2 $\frac{1}{4}$	31.870
"	2 $\frac{1}{4}$	20.620	"	3 $\frac{1}{4}$	37.910	"	2 $\frac{1}{2}$	35.410
"	2 $\frac{1}{2}$	21.770	"	3 $\frac{1}{2}$	40.830	"	2 $\frac{3}{4}$	38.950
"	2 $\frac{3}{8}$	22.910	3 $\frac{3}{4}$	1 $\frac{1}{4}$	3.125	"	3	42.500
"	2 $\frac{1}{2}$	24.060	"	1 $\frac{1}{2}$	6.250	"	3 $\frac{1}{4}$	46.030
"	2 $\frac{3}{4}$	25.200	"	1 $\frac{3}{4}$	9.375	"	3 $\frac{1}{2}$	49.570
3	1 $\frac{1}{2}$	2.500	"	1	12.500	"	3 $\frac{3}{4}$	53.120
"	1 $\frac{3}{8}$	5.000	"	1 $\frac{1}{4}$	15.620	"	4	56.660
"	1 $\frac{1}{4}$	7.500	"	1 $\frac{1}{2}$	18.750	"	4 $\frac{1}{4}$	60.200
"	1	10.000	"	1 $\frac{3}{4}$	21.870	4 $\frac{1}{2}$	1 $\frac{1}{4}$	3.750
"	1 $\frac{1}{4}$	12.500	"	2	25.000	"	1 $\frac{1}{2}$	7.500
"	1 $\frac{1}{2}$	15.000	"	2 $\frac{1}{4}$	28.120	"	1 $\frac{3}{4}$	11.250
"	1 $\frac{3}{4}$	17.500	"	2 $\frac{1}{2}$	31.250	"	1	15.000
"	2	20.000	"	2 $\frac{3}{4}$	34.370	"	1 $\frac{1}{4}$	18.750
"	2 $\frac{1}{4}$	22.500	"	3	37.500	"	1 $\frac{1}{2}$	22.500
"	2 $\frac{1}{2}$	25.000	"	3 $\frac{1}{4}$	40.620	"	1 $\frac{3}{4}$	26.250
"	2 $\frac{3}{4}$	27.500	"	3 $\frac{1}{2}$	43.750	"	2	30.000
"	3	30.000	"	3 $\frac{3}{4}$	46.860	"	2 $\frac{1}{4}$	33.750
3 $\frac{1}{4}$	1 $\frac{1}{4}$	2.708	4	1 $\frac{1}{4}$	3.330	"	2 $\frac{1}{2}$	37.500
"	1 $\frac{1}{2}$	5.416	"	1 $\frac{1}{2}$	6.660	"	2 $\frac{3}{4}$	41.250
"	1 $\frac{3}{4}$	8.124	"	1 $\frac{3}{4}$	10.000	"	3	45.000
"	1	10.830	"	1	13.330	"	3 $\frac{1}{4}$	48.750
"	1 $\frac{1}{4}$	13.500	"	1 $\frac{1}{4}$	16.660	"	3 $\frac{1}{2}$	52.500
"	1 $\frac{1}{2}$	16.250	"	1 $\frac{1}{2}$	20.000	"	3 $\frac{3}{4}$	56.250
"	1 $\frac{3}{4}$	18.950	"	1 $\frac{3}{4}$	23.330	"	4	60.000

Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.
4 $\frac{1}{2}$	4 $\frac{1}{4}$	63.750	5 $\frac{1}{4}$	1 $\frac{3}{4}$	8.753	5 $\frac{3}{4}$	1 $\frac{1}{4}$	4.788
"	4 $\frac{1}{2}$	67.500	"	1 $\frac{3}{4}$	13.130	"	1 $\frac{1}{2}$	9.587
4 $\frac{3}{4}$	4 $\frac{1}{2}$	3.953	"	1	17.500	"	1 $\frac{3}{4}$	14.370
"	4 $\frac{3}{4}$	7.910	"	1 $\frac{1}{4}$	21.870	"	1	19.160
"	4 $\frac{3}{4}$	11.860	"	1 $\frac{1}{2}$	26.250	"	1 $\frac{1}{4}$	23.950
"	1	15.830	"	1 $\frac{1}{2}$	30.620	"	1 $\frac{1}{2}$	28.750
"	1 $\frac{1}{4}$	19.760	"	2	35.000	"	1 $\frac{3}{4}$	33.540
"	1 $\frac{1}{2}$	23.750	"	2 $\frac{1}{4}$	39.370	"	2	38.330
"	1 $\frac{3}{4}$	27.700	"	2 $\frac{1}{2}$	43.750	"	2 $\frac{1}{4}$	43.120
"	2	31.670	"	2 $\frac{3}{4}$	48.110	"	2 $\frac{1}{2}$	47.910
"	2 $\frac{1}{4}$	35.620	"	3	52.500	"	2 $\frac{3}{4}$	52.700
"	2 $\frac{1}{2}$	39.580	"	3 $\frac{1}{4}$	56.680	"	3	57.500
"	2 $\frac{3}{4}$	43.540	"	3 $\frac{1}{2}$	61.250	"	3 $\frac{1}{4}$	62.300
"	3	47.500	"	3 $\frac{3}{4}$	65.620	"	3 $\frac{1}{2}$	67.080
"	3 $\frac{1}{4}$	51.460	"	4	70.000	"	3 $\frac{3}{4}$	71.860
"	3 $\frac{1}{2}$	55.410	"	4 $\frac{1}{4}$	74.370	"	4	76.650
"	3 $\frac{3}{4}$	59.370	"	4 $\frac{1}{2}$	78.750	"	4 $\frac{1}{4}$	81.450
"	4	63.330	"	4 $\frac{3}{4}$	83.110	"	4 $\frac{1}{2}$	86.240
"	4 $\frac{1}{4}$	67.290	"	5	87.500	"	4 $\frac{3}{4}$	91.030
"	4 $\frac{1}{2}$	71.250	"	5 $\frac{1}{4}$	91.860	"	5	95.820
"	4 $\frac{3}{4}$	75.200	5 $\frac{1}{2}$	1 $\frac{1}{4}$	4.587	"	5 $\frac{1}{4}$	100.600
5	1 $\frac{1}{4}$	4.166	"	1 $\frac{1}{2}$	9.164	"	5 $\frac{1}{2}$	105.400
"	1 $\frac{1}{2}$	8.330	"	1 $\frac{3}{4}$	13.750	"	5 $\frac{3}{4}$	119.700
"	1 $\frac{3}{4}$	12.500	"	1	18.330	"	6	10.000
"	1	16.660	"	1 $\frac{1}{4}$	22.900	"	1	20.000
"	1 $\frac{1}{4}$	20.830	"	1 $\frac{1}{2}$	27.500	"	1 $\frac{1}{2}$	30.000
"	1 $\frac{1}{2}$	25.000	"	1 $\frac{3}{4}$	32.080	"	2	40.000
"	1 $\frac{3}{4}$	29.160	"	2	36.660	"	2 $\frac{1}{2}$	50.000
"	2	33.330	"	2 $\frac{1}{4}$	41.250	"	3	60.000
"	2 $\frac{1}{4}$	37.500	"	2 $\frac{1}{2}$	45.830	"	3 $\frac{1}{2}$	70.000
"	2 $\frac{1}{2}$	41.660	"	2 $\frac{3}{4}$	50.310	"	4	80.000
"	2 $\frac{3}{4}$	45.830	"	3	55.000	"	4 $\frac{1}{2}$	90.000
"	3	50.000	"	3 $\frac{1}{4}$	59.570	"	5	100.000
"	3 $\frac{1}{4}$	54.160	"	3 $\frac{1}{2}$	64.160	"	5 $\frac{1}{2}$	110.000
"	3 $\frac{3}{4}$	58.330	"	3 $\frac{3}{4}$	68.740	"	6	120.000
"	4	62.500	"	4	73.330	6 $\frac{1}{2}$	1 $\frac{1}{2}$	10.830
"	4 $\frac{1}{4}$	66.660	"	4 $\frac{1}{2}$	77.910	"	1	21.660
"	4 $\frac{1}{2}$	70.830	"	4 $\frac{3}{4}$	82.500	"	1 $\frac{1}{2}$	32.500
"	4 $\frac{3}{4}$	75.000	"	5	87.080	"	2	43.330
"	5	79.160	"	5 $\frac{1}{4}$	91.560	"	2 $\frac{1}{2}$	54.160
"	5 $\frac{1}{4}$	83.330	"	5 $\frac{1}{2}$	96.240	"	3	65.000
5 $\frac{1}{4}$	1 $\frac{1}{4}$	4.376	"	5 $\frac{1}{2}$	100.600	"	3 $\frac{1}{2}$	75.830

Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.
6½	4	86.66	8	4	106.60	9	8½	255.00
"	4½	97.50	"	4½	120.00	"	9	270.00
"	5	108.30	"	5	133.30	9½	½	15.83
"	5½	119.10	"	5½	146.60	"	1	31.66
"	6	130.00	"	6	160.00	"	1½	47.50
"	6½	140.80	"	6½	173.30	"	2	63.33
7	½	11.66	"	7	186.60	"	2½	79.16
"	1	23.33	"	7½	200.00	"	3	95.00
"	1½	35.00	"	8	213.30	"	3½	110.80
"	2	46.66	8½	½	14.16	"	4	126.60
"	2½	58.33	"	1	28.33	"	4½	142.50
"	3	70.00	"	1½	42.48	"	5	158.30
"	3½	81.66	"	2	56.66	"	5½	174.10
"	4	93.33	"	2½	70.83	"	6	190.00
"	4½	105.00	"	3	85.00	"	6½	205.80
"	5	116.60	"	3½	99.16	"	7	221.60
"	5½	128.30	"	4	113.30	"	7½	237.60
"	6	140.00	"	4½	127.50	"	8	253.30
"	6½	151.60	"	5	141.60	"	8½	269.10
"	7	163.30	"	5½	155.80	"	9	285.00
7½	½	12.50	"	6	170.00	"	9½	300.80
"	1	25.00	"	6½	184.10	10	½	16.66
"	1½	37.50	"	7	198.30	"	1	33.33
"	2	50.00	"	7½	212.50	"	1½	50.00
"	2½	62.50	"	8	226.60	"	2	66.66
"	3	75.00	"	8½	240.70	"	2½	83.33
"	3½	87.50	9	½	15.00	"	3	100.00
"	4	100.00	"	1	30.00	"	3½	116.60
"	4½	112.50	"	1½	45.00	"	4	133.30
"	5	125.00	"	2	60.00	"	4½	150.00
"	5½	137.50	"	2½	75.00	"	5	166.60
"	6	150.00	"	3	90.00	"	5½	183.30
"	6½	162.50	"	3½	105.00	"	6	200.00
"	7	175.00	"	4	120.00	"	6½	216.60
"	7½	187.50	"	4½	135.00	"	7	233.30
8	½	13.33	"	5	150.00	"	7½	250.00
"	1	26.66	"	5½	165.00	"	8	266.60
"	1½	40.00	"	6	180.00	"	8½	283.30
"	2	53.33	"	6½	195.00	"	9	300.00
"	2½	66.66	"	7	210.00	"	9½	316.60
"	3	80.00	"	7½	225.00	"	10	333.30
"	3½	93.33	"	8	240.00	10½	½	17.50

Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.	Breadth in inches.	Thickness in inches.	Weight in lbs.
10½	1	35.00	11	1½	55.00	11½	1½	57.50
"	1½	52.50	"	2	73.33	"	2	76.66
"	2	70.00	"	2½	91.56	"	2½	95.83
"	2½	87.50	"	3	110.00	"	3	115.00
"	3	105.00	"	3½	128.30	"	3½	134.10
"	3½	122.50	"	4	146.60	"	4	153.30
"	4	140.00	"	4½	165.00	"	4½	172.50
"	4½	157.50	"	5	183.30	"	5	191.60
"	5	175.00	"	5½	201.60	"	5½	210.80
"	5½	192.50	"	6	220.00	"	6	230.00
"	6	210.00	"	6½	238.30	"	6½	249.10
"	6½	227.50	"	7	256.60	"	7	268.30
"	7	245.00	"	7½	275.00	"	7½	287.50
"	7½	262.50	"	8	293.30	"	8	306.60
"	8	280.00	"	8½	311.60	"	8½	325.80
"	8½	297.50	"	9	330.00	"	9	345.00
"	9	315.00	"	9½	348.30	"	9½	364.10
"	9½	332.50	"	10	366.60	"	10	383.30
"	10	350.00	"	10½	385.00	"	10½	402.50
"	10½	367.50	"	11	403.30	"	11	421.60
11	10½	367.50	"	11½	421.60	"	11½	440.70
"	1	18.33	11½	1½	19.16	"	11½	440.70
"	1	36.66	"	1	38.33	12	12	480.00

WEIGHT OF A LINEAL FOOT OF ROLLED ROUND IRON IN POUNDS AVOIRDUPOIS.

(480 pounds per cubic foot.)

Diameter in inches.	Weight in lbs.	Diameter in inches.	Weight in lbs.	Diameter in inches.	Weight in lbs.	Diameter in inches.	Weight in lbs.
$\frac{1}{16}$	0.010	$\frac{23}{8}$	14.77	$\frac{55}{8}$	82.79	$\frac{87}{8}$	206.2
$\frac{1}{8}$	0.041	$\frac{21}{4}$	16.36	$\frac{53}{8}$	86.52	9	212.2
$\frac{3}{16}$	0.091	$\frac{23}{8}$	18.04	$\frac{51}{8}$	90.34	$\frac{91}{8}$	218.0
$\frac{1}{4}$	0.163	$\frac{25}{8}$	19.80	6	94.26	$\frac{93}{8}$	223.9
$\frac{5}{16}$	0.255	$\frac{27}{8}$	21.64	$\frac{61}{8}$	98.18	$\frac{95}{8}$	230.1
$\frac{3}{8}$	0.368	3	23.56	$\frac{63}{8}$	102.20	$\frac{97}{8}$	236.2
$\frac{7}{16}$	0.501	$\frac{31}{8}$	25.56	$\frac{65}{8}$	106.40	$\frac{99}{8}$	242.5
$\frac{1}{2}$	0.655	$\frac{33}{8}$	27.64	$\frac{67}{8}$	110.60	$\frac{101}{8}$	248.9
$\frac{9}{16}$	0.828	$\frac{35}{8}$	29.82	$\frac{69}{8}$	114.90	$\frac{103}{8}$	255.2
$\frac{5}{8}$	1.022	$\frac{37}{8}$	32.07	$\frac{71}{8}$	119.30	10	261.7
$\frac{11}{16}$	1.237	$\frac{39}{8}$	34.39	$\frac{73}{8}$	123.70	$10\frac{1}{8}$	268.4
$\frac{3}{4}$	1.473	$\frac{41}{8}$	36.81	7	128.30	$10\frac{1}{4}$	275.0
$\frac{13}{16}$	1.728	$\frac{43}{8}$	39.30	$\frac{75}{8}$	132.90	$10\frac{3}{8}$	281.8
$\frac{7}{8}$	2.004	4	41.88	$\frac{77}{8}$	137.60	$10\frac{1}{2}$	288.6
$\frac{15}{16}$	2.301	$\frac{45}{8}$	44.57	$\frac{79}{8}$	142.30	$10\frac{5}{8}$	295.6
1	2.618	$\frac{47}{8}$	47.28	$\frac{81}{8}$	147.30	$10\frac{3}{4}$	302.5
$1\frac{1}{8}$	3.310	$\frac{49}{8}$	50.10	$\frac{83}{8}$	152.20	$10\frac{7}{8}$	309.5
$1\frac{1}{4}$	4.094	$\frac{51}{8}$	53.02	$\frac{85}{8}$	157.20	11	316.8
$1\frac{3}{8}$	4.950	$\frac{53}{8}$	56.03	$\frac{87}{8}$	162.40	$11\frac{1}{8}$	323.9
$1\frac{1}{2}$	5.885	$\frac{55}{8}$	59.05	8	167.50	$11\frac{1}{4}$	331.3
$1\frac{5}{8}$	6.911	$\frac{57}{8}$	62.17	$8\frac{1}{8}$	172.80	$11\frac{3}{8}$	338.7
$1\frac{3}{4}$	8.018	5	65.49	$8\frac{1}{4}$	178.20	$11\frac{1}{2}$	346.2
$1\frac{7}{8}$	9.205	$5\frac{1}{8}$	68.71	$8\frac{3}{8}$	183.60	$11\frac{5}{8}$	353.7
2	10.470	$5\frac{1}{4}$	72.13	$8\frac{1}{2}$	189.10	$11\frac{3}{4}$	361.5
$2\frac{1}{8}$	11.820	$5\frac{3}{8}$	75.65	$8\frac{3}{4}$	194.80	$11\frac{7}{8}$	369.1
$2\frac{1}{4}$	13.250	$5\frac{1}{2}$	79.17	$8\frac{7}{8}$	200.40	12	376.9

BOLTS, NUTS, AND HEADS.

(Whitworth's Proportions.)

Weight in lbs. of Heads and Nuts.

Diameter of bolt in in.	Hexagonal.		Square.		Hexagonal.		Square.	
	Head.	Nut.	Head.	Nut.	Two Heads.	Head & Nut.	Two Heads.	Head & Nut.
$\frac{1}{16}$	0.008	0.005	0.022	0.019	0.017	0.013	0.044	0.041
$\frac{1}{8}$	0.014	0.007	0.027	0.021	0.029	0.022	0.055	0.048
$\frac{3}{16}$	0.029	0.017	0.061	0.049	0.057	0.046	0.122	0.110
$\frac{1}{4}$	0.059	0.040	0.069	0.050	0.119	0.101	0.138	0.119
$\frac{5}{16}$	0.068	0.041	0.104	0.076	0.136	0.109	0.208	0.181
$\frac{3}{8}$	0.104	0.065	0.157	0.118	0.208	0.169	0.315	0.276
$\frac{7}{16}$	0.151	0.097	0.246	0.193	0.302	0.248	0.493	0.440
$\frac{1}{2}$	0.254	0.161	0.362	0.269	0.508	0.415	0.724	0.631
$\frac{5}{8}$	0.367	0.219	0.551	0.408	0.734	0.586	1.102	0.959
$\frac{3}{4}$	0.546	0.326	0.683	0.463	1.092	0.872	1.366	1.146
$\frac{7}{8}$	0.724	0.411	1.109	0.797	1.448	1.135	2.217	1.906
1	1.060	0.630	1.400	0.971	2.120	1.690	2.800	2.371
1 $\frac{1}{8}$	1.330	0.759	1.949	1.379	2.660	2.088	3.898	3.328
1 $\frac{1}{4}$	1.840	1.098	2.625	1.883	3.680	2.938	5.250	4.508
1 $\frac{3}{8}$	2.460	1.517	3.135	2.192	4.920	3.977	6.270	5.327
1 $\frac{1}{2}$	2.920	1.742	3.704	2.532	5.840	4.662	7.409	6.236
1 $\frac{3}{4}$	3.440	1.991	4.725	3.276	6.880	5.431	9.450	8.001
2	4.370	2.611	6.384	4.625	8.740	6.981	12.77	11.00
2 $\frac{1}{4}$	6.150	3.645	8.858	6.353	12.30	9.795	17.71	15.21
2 $\frac{1}{2}$	8.480	5.045	11.91	8.476	16.96	13.52	23.82	20.39
2 $\frac{3}{4}$	11.32	6.747	15.59	9.019	22.64	18.06	31.18	24.61
3	14.72	8.783	21.00	15.06	29.44	23.50	42.00	36.06

WEIGHT IN POUNDS OF ROUND IRON FOR

Diameter in inches.	Length in inches.									
	$\frac{1}{8}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	2	3
$\frac{1}{4}$	0.002	0.003	0.005	0.007	0.008	0.010	0.012	0.014	0.027	0.041
$\frac{5}{16}$	0.003	0.005	0.008	0.011	0.013	0.016	0.019	0.021	0.043	0.064
$\frac{3}{8}$	0.004	0.007	0.011	0.015	0.019	0.023	0.027	0.031	0.062	0.093
$\frac{7}{16}$	0.005	0.010	0.016	0.021	0.026	0.031	0.036	0.042	0.084	0.126
$\frac{1}{2}$	0.007	0.014	0.021	0.027	0.034	0.041	0.048	0.055	0.110	0.166
$\frac{9}{16}$	0.009	0.017	0.026	0.035	0.043	0.052	0.061	0.069	0.139	0.208
$\frac{5}{8}$	0.011	0.022	0.032	0.043	0.054	0.065	0.076	0.087	0.174	0.261
$\frac{3}{4}$	0.015	0.031	0.046	0.062	0.077	0.093	0.108	0.124	0.249	0.373
$\frac{7}{8}$	0.021	0.042	0.063	0.084	0.105	0.126	0.148	0.170	0.338	0.508
1	0.027	0.055	0.083	0.110	0.138	0.165	0.193	0.221	0.442	0.663
$1\frac{1}{8}$	0.035	0.070	0.105	0.140	0.185	0.210	0.245	0.280	0.560	0.840
$1\frac{1}{4}$	0.043	0.087	0.131	0.173	0.217	0.262	0.304	0.347	0.695	1.043
$1\frac{3}{8}$	0.053	0.104	0.157	0.209	0.261	0.314	0.366	0.418	0.836	1.255
$1\frac{1}{2}$	0.062	0.124	0.186	0.249	0.311	0.373	0.435	0.497	0.995	1.493
$1\frac{3}{4}$	0.072	0.143	0.215	0.287	0.358	0.430	0.502	0.584	1.168	1.752
$1\frac{7}{8}$	0.084	0.168	0.253	0.337	0.421	0.506	0.590	0.677	1.354	2.032
2	0.097	0.194	0.291	0.389	0.486	0.583	0.680	0.778	1.555	2.333
$2\frac{1}{4}$	0.111	0.221	0.332	0.442	0.553	0.663	0.774	0.884	1.770	2.654
$2\frac{1}{2}$	0.140	0.280	0.420	0.560	0.700	0.840	0.980	1.120	2.240	3.360
$2\frac{3}{4}$	0.174	0.347	0.521	0.695	0.869	1.042	1.216	1.390	2.781	4.172
$2\frac{7}{8}$	0.209	0.418	0.627	0.836	1.045	1.254	1.463	1.673	3.346	5.019
3	0.250	0.500	0.750	1.000	1.250	1.500	1.750	1.990	3.981	5.972

EXAMPLE.—Required, the weight of a bolt $1\frac{1}{4}$ inches diameter, 4 inches between inside of head and nut.

Weight of bolt = 1.39

Weight of square head = 1.40

Weight of hexagonal nut = 1.06 taken as a hexagonal head

Ans. 3.85 lbs.

BOLTS, ETC., BETWEEN HEAD AND NUT.

Diameter in inches.	Length in inches.								
	4	5	6	7	8	9	10	11	12
$\frac{1}{16}$	0.055	0.069	0.082	0.096	0.110	0.124	0.137	0.151	0.165
$\frac{1}{8}$	0.086	0.107	0.128	0.150	0.171	0.192	0.214	0.235	0.257
$\frac{3}{16}$	0.124	0.155	0.186	0.217	0.248	0.279	0.311	0.342	0.373
$\frac{1}{4}$	0.167	0.209	0.251	0.293	0.335	0.377	0.419	0.461	0.503
$\frac{5}{16}$	0.221	0.276	0.331	0.386	0.442	0.497	0.552	0.607	0.663
$\frac{3}{8}$	0.277	0.347	0.416	0.486	0.555	0.624	0.694	0.763	0.833
$\frac{7}{16}$	0.347	0.434	0.521	0.608	0.695	0.782	0.869	0.956	1.043
$\frac{1}{2}$	0.497	0.622	0.746	0.871	0.995	1.119	1.244	1.368	1.493
$\frac{9}{16}$	0.677	0.846	1.016	1.185	1.354	1.524	1.693	1.862	2.032
$\frac{5}{8}$	0.884	1.105	1.326	1.548	1.769	1.990	2.211	2.432	2.654
$\frac{11}{16}$	1.120	1.400	1.680	1.960	2.240	2.520	2.800	3.080	3.360
$\frac{3}{4}$	1.390	1.738	2.085	2.433	2.781	3.128	3.476	3.823	4.172
$\frac{13}{16}$	1.673	2.091	2.510	2.928	3.346	3.765	4.182	4.601	5.019
$\frac{7}{8}$	1.990	2.488	2.985	3.483	3.981	4.478	4.976	5.473	5.972
$\frac{15}{16}$	2.336	2.920	3.504	4.088	4.673	5.257	5.841	6.425	7.010
$1\frac{1}{16}$	2.709	3.386	4.064	4.741	5.418	6.096	6.773	7.450	8.128
$1\frac{1}{8}$	3.111	3.888	4.666	5.334	6.221	6.999	7.777	8.547	9.333
$1\frac{1}{4}$	3.538	4.423	5.307	6.192	7.077	7.961	8.846	9.730	10.610
$1\frac{3}{8}$	4.480	5.600	6.720	7.840	8.960	10.080	11.200	12.320	13.440
$1\frac{1}{2}$	5.562	6.953	8.343	9.734	11.120	12.510	13.910	15.290	16.690
$1\frac{3}{4}$	6.692	8.365	10.040	11.710	13.380	15.060	16.730	18.400	20.070
2	7.962	9.953	11.940	13.930	15.920	17.910	19.910	21.890	23.890

WEIGHT OF MATERIALS USED IN BUILDING.

(Per square foot from one inch thickness to a cubic foot.)

Stones, Earths, &c.

Thickness in inches.	Asphaltum, average.	Basalts, average.	Brick.			Plaster of Paris.	Common gravel.	Limestone.	Marble, average.	Mortar.	Mud.	Oyster shell.
			Average.	Fire.	In cement or mortar.							
1	6.58	14.58	8.50	11.41	9.33	6.12	9.08	16.5	14.08	8.16	8.5	10.83
2	13.16	29.16	17.00	22.83	18.66	12.25	18.16	33.0	28.16	16.33	17.0	21.66
3	19.74	43.74	25.50	34.24	28.00	18.36	27.24	49.5	42.25	24.50	25.5	32.49
4	26.32	58.32	34.00	45.66	37.33	24.50	36.33	66.0	56.32	32.66	34.0	43.33
5	32.90	72.90	42.50	57.08	46.66	30.61	45.41	82.5	70.40	40.83	42.5	54.16
6	39.48	87.48	51.00	68.50	56.00	36.74	54.50	99.0	84.48	49.00	51.0	65.00
7	46.06	102.06	59.50	80.00	65.33	42.86	63.60	115.5	98.56	57.16	59.5	75.83
8	52.64	116.64	68.00	91.32	74.66	49.00	72.66	132.0	112.64	65.32	68.0	86.66
9	59.22	131.22	76.50	102.75	84.00	55.10	81.75	148.5	126.72	72.50	76.5	97.50
10	65.80	145.80	85.00	114.16	93.33	61.23	90.83	165.0	140.80	81.66	85.0	108.33
11	72.38	160.38	93.50	125.60	102.66	67.35	99.13	181.5	154.90	89.82	93.5	119.16
12	79.00	175.00	102.00	137.00	112.00	73.50	109.00	198.0	169.00	98.00	102.0	130.00

Stones, Earths, &c.

Thickness in inches.	Portland cement.	Chalk.	Clay.	Clay with gravel.	Concrete, average.	Earth, common soil.	Glass, (window.)	Common sand.	Slate.	Granite.		Rain water.
										Patapsco.	Susquehanna.	
1	6.75	11.16	10.0	12.91	10.41	11.41	13.75	8.66	12.25	13.75	14.08	5.21
2	13.50	22.33	20.0	25.82	20.83	22.83	27.50	17.33	24.50	27.50	28.16	10.42
3	20.25	33.50	30.0	38.73	31.25	34.25	41.25	26.00	36.75	41.25	42.24	15.62
4	27.00	44.66	40.0	51.64	41.66	45.66	55.00	34.66	49.00	55.00	56.32	20.83
5	33.75	55.83	50.0	64.55	52.08	57.08	68.75	43.33	61.25	68.75	70.40	26.04
6	40.50	67.00	60.0	77.46	64.50	68.50	82.50	52.00	73.50	82.50	84.48	31.24
7	47.25	78.16	70.0	90.37	73.00	80.00	96.25	60.66	85.75	96.25	98.56	36.45
8	54.00	89.33	80.0	103.28	83.32	91.32	110.00	69.22	98.00	110.00	112.64	41.66
9	60.75	100.50	90.0	116.19	93.75	102.75	123.75	80.00	110.25	123.75	126.72	46.87
10	67.50	111.66	100.0	129.10	104.16	114.16	137.50	86.66	122.50	137.50	140.80	52.08
11	74.25	122.83	110.0	142.01	114.57	125.57	150.25	95.32	134.75	150.25	154.88	57.28
12	81.00	134.00	120.0	155.00	125.00	137.00	165.00	104.00	147.00	165.00	169.00	62.50

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